12

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CHUTE SPILLWAY FOR COWANESQUE DAM
COWANESQUE RIVER, PENNSYLVANIA

Hydraulic Model Investigation

by

Bobby P. Fletcher

Hydraulics Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

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Tests to investigate the hydraulic performance of the chute spillway for Cowanesque Lake were conducted using a 1:60-scale model. Particular emphasis was placed on the development of a structure that would convey flow through the chute with a minimum of turbulence and waves.

Flow approaching the structure from the left created a severe flow contraction at the left spillway abutment which became progressively more severe as the discharge increased. The flow contraction generated a (Continued)

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20. ABSTRACT (Continued)

standing wave that overtopped the chute sidewall. Flow distribution at the downstream end of the chute was unsymmetrical. Lateral flow from the left side of the exit area tended to concentrate the jet emerging from the chute along the right side of the exit channel, and velocities as high as 55 fps were measured.

The flow contraction at the left abutment, unsymmetrical flow distribution in the chute, and the standing waves in the chute were attenuated by installing an elliptical wall, deepening the approach along the inside of the wall at the left abutment, and reducing the rate of convergence of the chute sidewalls. The minimum stone size (d_{50} = 18 in.) required for protection of the elliptical wall was determined from the model. The elliptical wall also increased the capacity of the structure, permitting passage of 224,000 cfs at a pool elevation of 1146.1.

(d sub 50)

Economically, it was desirable to move the intake tower closer to the dam. Model tests indicated that the intake tower could be moved 150 ft closer to the dam embankment without adversely affecting flow conditions.

Due to structural and foundation conditions, the spillway structure was rotated 0°-55'-37" in a southeasterly direction about a point near the downstream end of the chute, and 90 ft of the downstream end of the left chute sidewall was removed. No significant change in the hydraulic performance of the structure was observed.

Severe currents, waves, and turbulence were observed along the right side of the exit channel for spillway discharges above 50,000 cfs. Velocities and wave heights along the left side of the exit area (dam embankment) were within acceptable limits for the range of anticipated discharges.

Model tests indicated that a stilling basin located at the downstream end of the chute would improve the performance of flow from the chute and in the exit channel. However, due to economic and structural considerations, a stilling basin was not considered feasible, and only preliminary stilling basin tests were conducted.

PREFACE

The model study of the chute spillway for Cowanesque Lake was authorized by the Office, Chief of Engineers, U. S. Army, on 12 March 1974, at the request of the U. S. Army Engineer District, Baltimore.

The study was conducted during the period March 1974 to July 1975 in the Hydraulics Laboratory of the U. S. Army Engineer Waterways Experiment Station (WES) under the direction of Messrs. H. B. Simmons, Chief of the Hydraulics Laboratory, and J. L. Grace, Chief of the Structures Division. The tests were conducted by Messrs. B. P. Fletcher, C. H. Tate, Jr., E. D. Rothwell, and B. Perkins under the direct supervision of Mr. J. P. Bohan, Chief of the Spillways and Channels Branch. This report was prepared by Mr. Fletcher.

During the course of the investigation, Messrs. S. Powell, R. Beane, W. McIntosh, and E. E. Eiker (formerly with Baltimore District) of the Office, Chief of Engineers; W. D. Stockman and B. Blackwell of the U. S. Army Engineer Division, North Atlantic; and C. E. Shores, C. R. Strong, R. E. Spath, E. Marcinski, N. Zweig, F. de la Sierra, and P. Hart of the Baltimore District visited WES to discuss test results and correlate these results with design studies.

Directors of the WES during the conduct of the study and the preparation and publication of this report were COL G. H. Hilt, CE, and COL John L. Cannon, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	25.4	millimetres
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
square miles	2.589988	square kilometres
acres	4046.856	square metres
acre-feet	1233.482	cubic metres
feet per second	0.3048	metres per second
cubic feet per second	0.02831685	cubic metres per second
degrees (angular)	0.01745329	radians

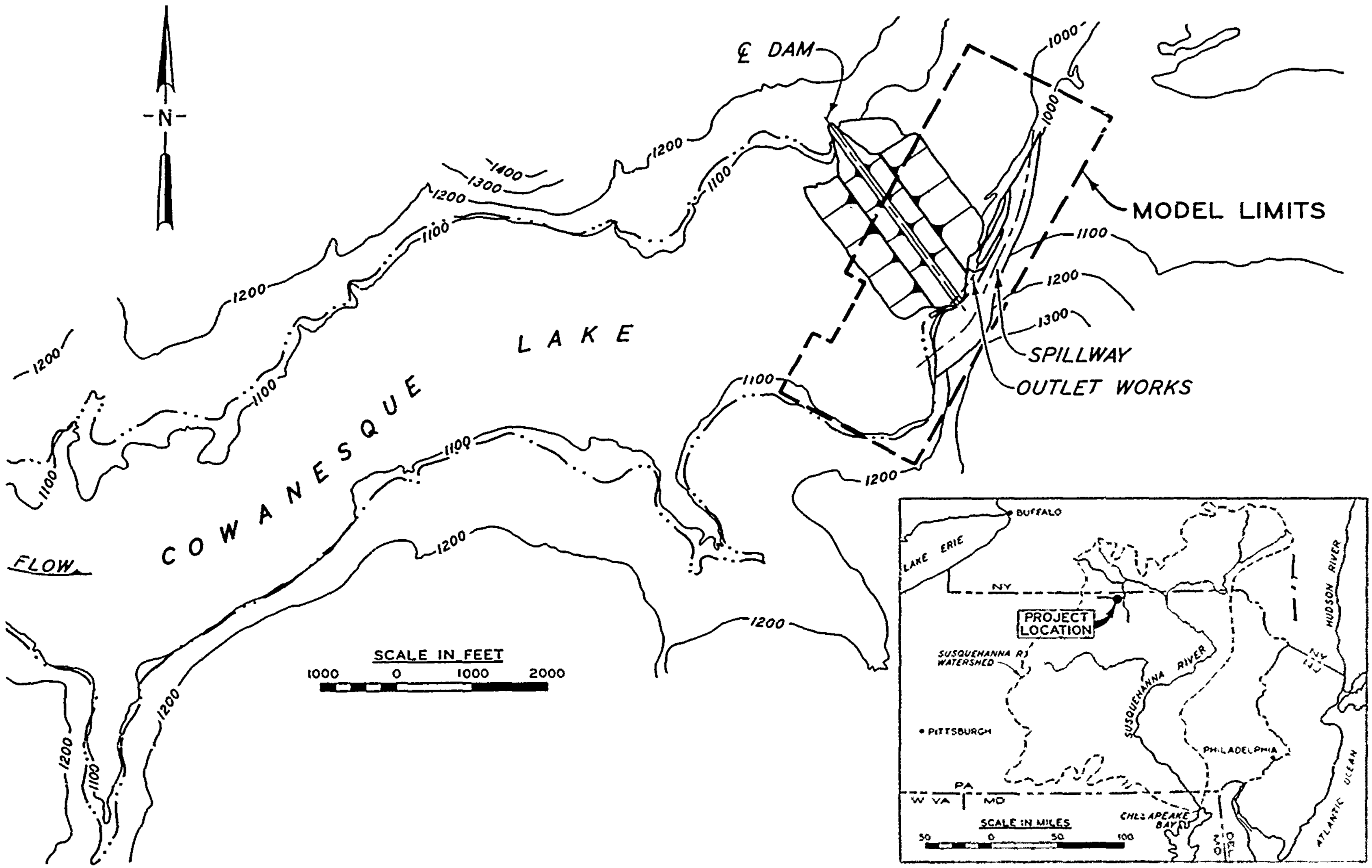

Figure 1. Project plan and vicinity map

CHUTE SPILLWAY FOR COWANESQUE DAM

COWANESQUE RIVER, PENNSYLVANIA

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. The Cowanesque Lake damsite (Figure 1) is located on the Cowanesque River, about 2.2 miles* upstream of its confluence with the Tioga River at Lawrenceville, Pennsylvania, and about 12 miles south of Corning, New York. The rolled earth and rock-filled dam will have total length of 3100 ft and a top elevation of 1151.0 ft** (151 ft above the streambed). The reservoir will provide a storage capacity of 89,000 acre-ft at the flood pool el 1117.0 to control runoff from a 298-square-mile drainage basin. At el 1117.0, the lake will extend 8 miles upstream and have a surface area of about 2060 acres. The uncontrolled 400-ft-long concrete weir with an ogee crest (el 1117.0) will be located at the right abutment and will pass the design flow of 224,000 cfs at a pool elevation of 1146.1.

2. The outlet works is located to the left of the chute spillway. Principal features of the outlet works include an approach channel, intake control tower, service bridge, tunnel, stilling basin, and exit channel. The conduit capacity with two gates fully open and the lake at spillway crest el 1117.0 is 9000 cfs. Flood control releases will be made through two 6- by 14-ft slide gates, and low-flow releases will be made through four inlet ports at various elevations. A general plan of the project and the portion simulated in the model is shown in Plate 1. Details of the structure are shown in Plate 2.

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

** All elevations (el) cited herein are in feet referred to mean sea level.

Purpose of Study

3. The spillway model tests were conducted to investigate the performance of the original design and develop, if necessary, a design that would provide satisfactory:

- a. Approach flow patterns and velocities.
- b. Flow characteristics at the abutments and crest.
- c. Wave patterns and water-surface profiles in the chute.
- d. Flow patterns, wave heights, and velocities in the exit channel and along the dam embankment.

The model was also used to determine the feasibility of a stilling basin at the end of the chute.

Definitions

4. To avoid confusion in reading this report and to prevent errors in changing from model to prototype values, all data herein are expressed in prototype terms. "Left" and "Right" indicate directions when looking downstream.

PART II: THE MODEL

Description

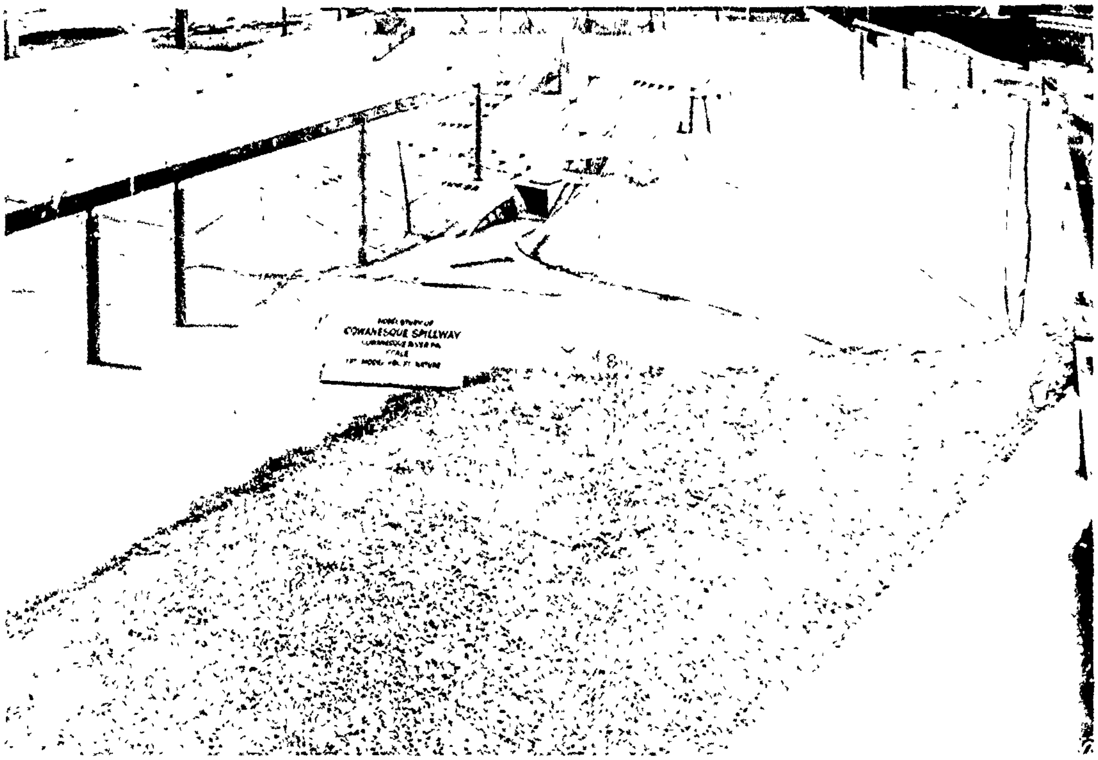
5. A 1:60-scale comprehensive chute spillway model (Figure 2 and Plate 1) was constructed to reproduce all topography and structures including the spillway and outlet works in an area extending 2000 ft upstream and 3000 ft downstream from the axis of the dam for a width of 2000 ft. The portions of the model representing the approach, exit, overbank areas, crest, and chute sidewalls were molded of cement mortar to sheet metal templates. The approach, exit, and overbank areas were given a brushed finish, and the crest and chute sidewalls were given a smooth finish. The chute invert and stilling basin were made of chemically treated wood to prevent expansion.

6. Water used in the operation of the model was supplied by a pump, and discharges were measured by means of venturi meters. Steel rails set to grade provided reference planes for measuring devices. Water-surface elevations were obtained with point gages. Velocities were measured with a pitot tube and by timing the movement of flotag and dye over measured distances. Current patterns were determined by observing the movement of dye injected into the water and confetti sprinkled on the water surface.

Interpretation of Model Results

7. The accepted equations of hydraulic similitude, based upon Froudian criteria, were used to express the mathematical relations between the dimensions and hydraulic quantities of the model and prototype. The general relations expressed in terms of the model scale or length ratio, L_r, are presented in the following tabulation:

Dimension	Ratio	Scale Relation
Length	L_r	1:60
Area	$A_r = L_r^2$	1:3,600



a. Looking downstream

b. Looking upstream

Figure 2. The 1:60 comprehensive model

Dimension	Ratio	Scale Relation
Velocity	$V_r = L_r^{1/2}$	1:7.746
Discharge	$Q_r = L_r^{5/2}$	1:27,886
Time	$T_r = L_r^{1/2}$	1:7.746

8. Model measurements of each dimension or variable can be transferred quantitatively to prototype equivalents by means of the preceding scale relations.

PART III: TESTS AND RESULTS

Presentation of Data

9. No attempt has been made to present the model tests and results in their chronological order. Instead, as each element of the structure is considered, all tests conducted thereon are described in detail. All data are presented in terms of prototype equivalents.

Approach Area

10. The model reproduced the approach for a distance of about 2000 ft upstream of the spillway (Plate 1 and Figure 2b). Plate 1 indicates the original and recommended location of the spillway. For either location, flow conditions in the approach area were similar and generally satisfactory (Photo 1), except in the immediate vicinity of the left abutment. Velocities in the approach for various discharges are shown in Plates 3-5. Tests were conducted to determine whether flow in the vicinity of the intake tower or spillway would be adversely affected by moving the tower closer to the dam embankment along the axis of the conduit. Several different locations were investigated, and it was determined that no adverse flow conditions were created until the tower was moved more than 150 ft downstream from the original location. Beyond this point, increased turbulence was created in the vicinity of the intake tower. Therefore, it was recommended that the tower be located no further than 150 ft downstream from its original location. The intake tower bridge piers had no adverse effects on flow condition. Griders supporting the bridge from the dam embankment to the intake tower may impede derbis floating on the surface at the design pool elevation and cause a structural problem.

Original Design (Abutments, Weir, and Chute)

11. A plan view of the original design is shown in Plate 2. Flow

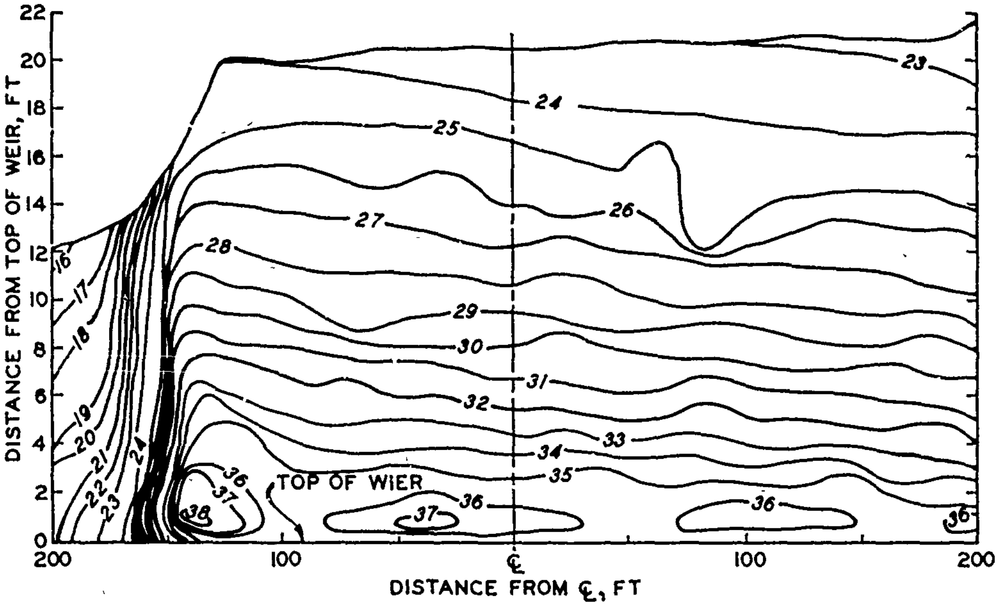
along the right abutment was satisfactory for the range of anticipated discharges. Flow approaching the spillway from the left experienced a flow contraction along the left abutment which became progressively more severe as the discharge was increased (Figure 3). Isovels and

Figure 3. Original design; discharge 200,000 cfs, pool el 1146.0

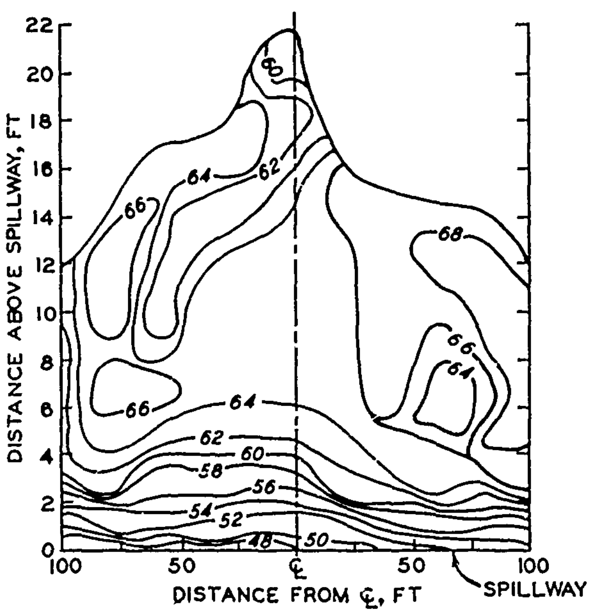
water-surface profiles along the weir axis and at sta 104+94.55 are shown in Figure 4. A comparison of the computed and model rating curves (Plate 6) indicates that the spillway is less efficient than anticipated due to the severe contraction at the left abutment. Basic model data are shown in Table 1. The magnitude of the abutment contraction coefficients is presented in Plate 7. These coefficients were computed based on discharges and heads indicated by the model and weir discharge coefficients shown in Hydraulic Design Chart (HDC)* 111-3.

12. The contraction at the left abutment also generates a severe standing wave that traveled obliquely down the chute overtopping the

* U. S. Army Corps of Engineers, "Hydraulic Design Criteria," prepared for the Office, Chief of Engineers, by the U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss., issued serially since 1952.



a. Sta 100+00 (weir axis)



b. Sta 104+94.55 (downstream end of convergence)

Figure 4. Original design with isovels and discharge 200,000 cfs

right wall near sta 109+00 (Photo 2). The standing wave was reflected off the right sidewall and entered the tailwater approximately 40 ft from the right side of the chute (Photo 3). Flow distribution at the downstream end of the chute was unsymmetrical. Water-surface profiles along the chute sidewalls for a discharge of 200,000 cfs are shown in Plate 8. The adverse conditions observed in the original design are a result of a combination of factors:

- a. The direction of the approaching flow and the resulting contraction at the left abutment.
- b. The ogee crest together with the battered chute sidewalls forms a rapid convergence as flow passes down the ogee crest and onto the chute. This creates a positive wave front that generates a cross wave pattern down the chute.
- c. The rapid rate of convergence of the chute and the decreasing Froude number in the direction of flow at the upper end of the chute.

Alternate and recommended left abutments

13. Several alternate abutment designs that include spur dikes and radial and elliptical walls were investigated to minimize the drawdown at the left abutment, standing waves, and uneven distribution of flow in the chute.

14. Various rock-lined spur dikes installed at the type 4 left abutment (Figure 5) did not improve flow conditions appreciably. A radial wall that consisted of a 96-ft radius and a 120-deg arc performed only slightly better than the spur dikes. An elliptical wall significantly improved the velocity distribution over the weir and reduced the magnitude of the standing waves traveling down the chute (Photo 4).

15. Due to the rapid increase in depth to solid rock near the left abutment, it was desirable to reduce the length and lateral dimensions of the wall. The upstream end of the parabolic wall was replaced by a circular curve, and the wall was moved 45 ft closer to the spillway. This resulted in a lower water depth at the upstream end of the wall, and the velocity measurements indicated a significant increase in the magnitude of the velocities and degree of turbulence in the vicinity

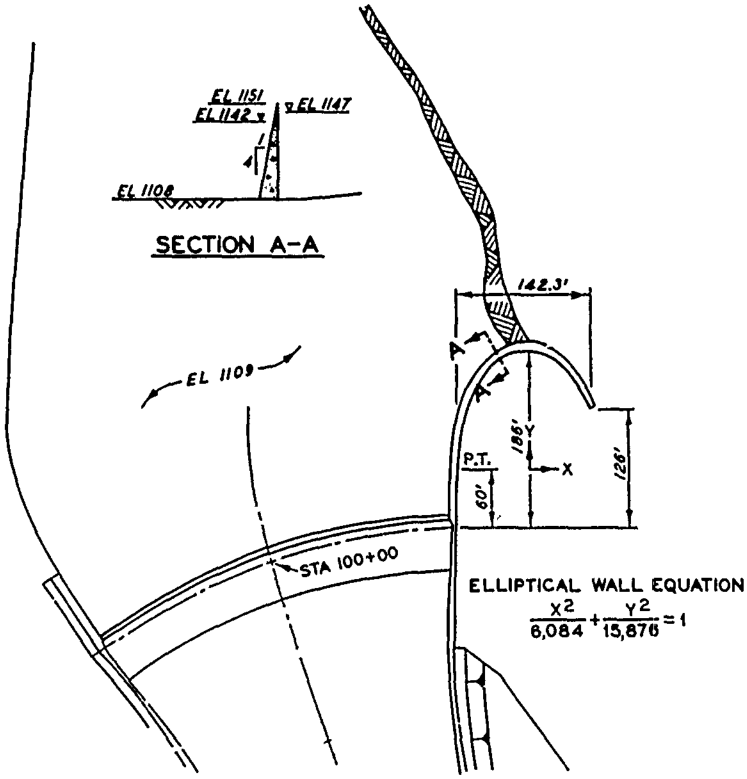
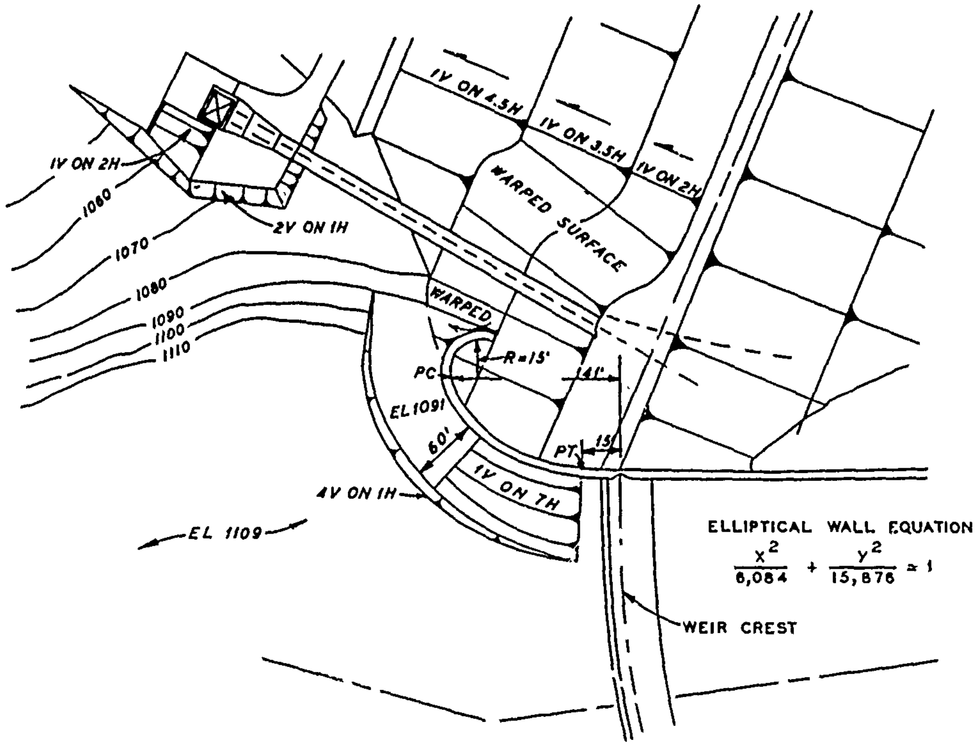


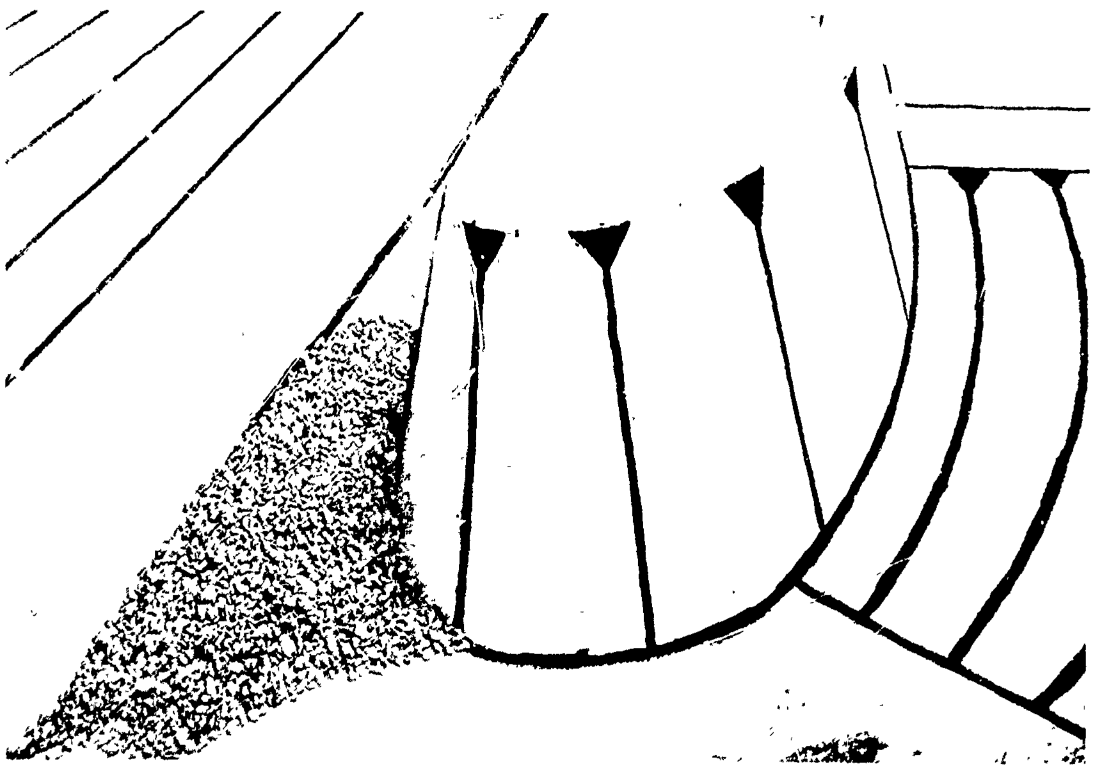
Figure 5. Type 4 left abutment

of the wall. Additional tests showed that excavating along the inside of the wall would reduce the magnitude of the velocities and degree of turbulence along the wall.

16. Tests of various left approach wall shapes and approach depths and widths resulted in the recommendation of the type 9 left abutment for prototype construction (Figure 6a). The type 9 design including the recommended crest and chute design (type 3) permitted passage of 224,000 cfs at a pool elevation of 1146.1 (Plate 6) and provided satisfactory flow conditions along the wall and over the crest (Figure 7). Deepening the approach along the inside of the wall reduced the Froude number, water-surface drawdown, and velocities at the left abutment (Figure 8). The maximum water-surface differential between the backside and inside of the wall occurred at the design discharge (224,000 cfs) at section A-A as shown in Figure 9. The abutment contraction coefficients are shown in Plate 7.



a. Plan view of wall



b. Stone required for protection of wall, d_{50} = 18 in.

Figure 6. Type 9 left abutment

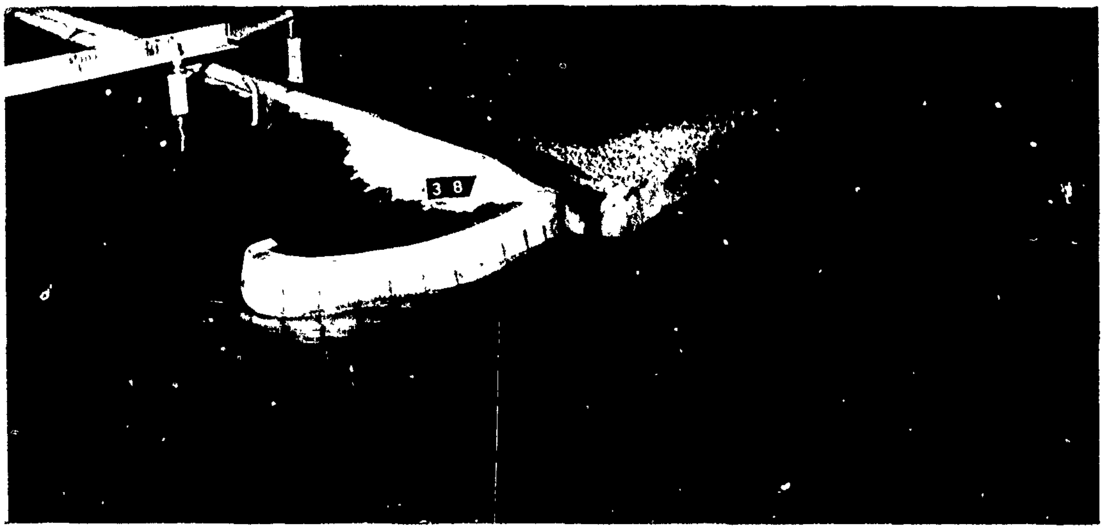


Figure 7. Flow conditions, recommended left abutment; discharge 224,000 cfs, pool el 1146.1

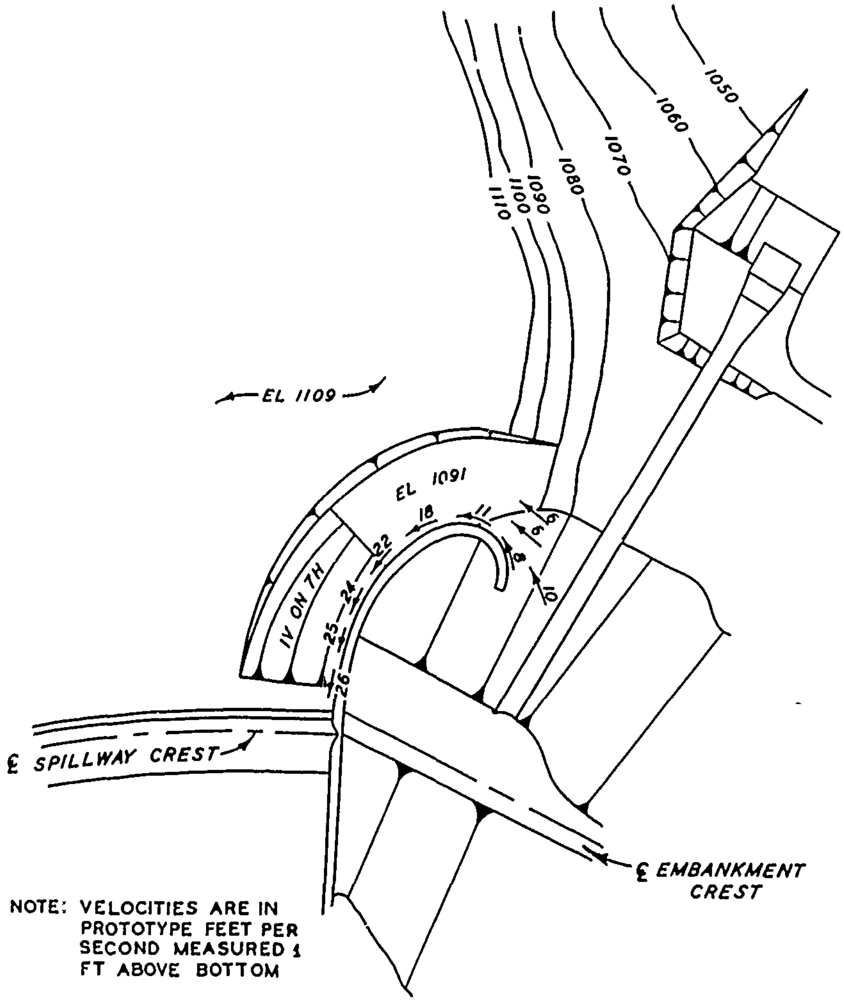


Figure 8. Type 9 left abutment, velocities and current directions; discharge 224,000 cfs, pool el 1146.1

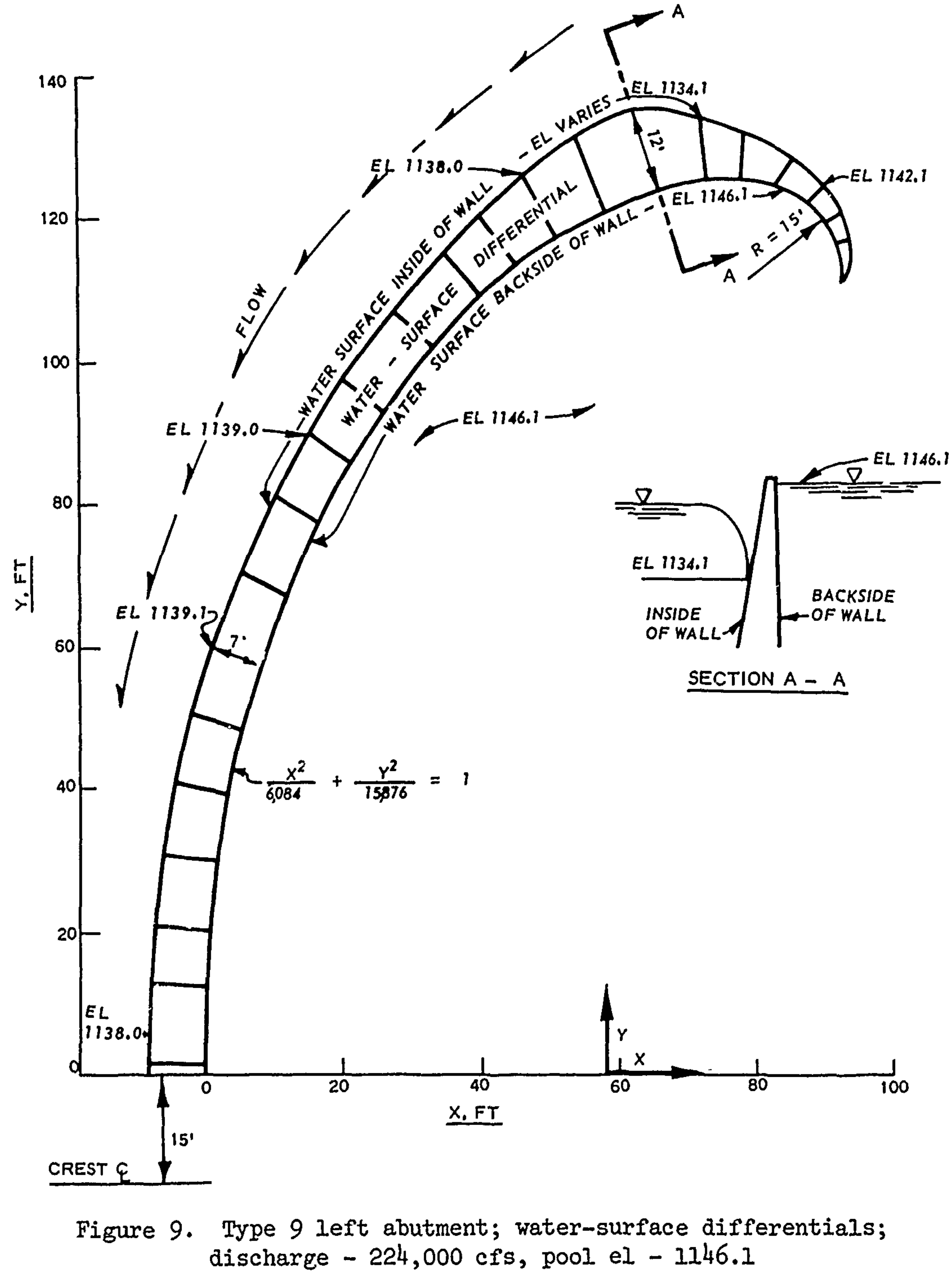


Figure 9. Type 9 left abutment; water-surface differentials; discharge - 224,000 cfs, pool el - 1146.1

17. Tests were conducted to determine the stone size required for protection of the dam embankment in the vicinity of the upstream end of the left abutment. The proposed riprap protection is shown in Figure 6b. The model indicated that the following gradation (average rock diameter = 18 in.) was adequate for protection with discharges as high as 224,000 cfs:

Rock Diameter, in.	Percent Finer
33	100
28	50-85
18	15-50
8	0-15

Recommended crest and chute

18. The crest and chute were modified by decreasing the rate of convergence in an attempt to reduce the standing waves and improve the flow distribution in the chute. In order to reduce the rate of convergence and provide a suitable rock foundation for the chute sidewalls, the spillway was rotated 0°-55'-37" in a southeasterly direction about a point near the downstream end of the original chute design (Plate 1). Dimensions of the original and recommended designs (type 3) are shown in Plates 2 and 9, respectively. The recommended design involved lowering the rate of convergence of the sidewalls from ratio 5:1 to 13:1 (chute length divided by one-half total lateral convergence). The widths of the original crest and downstream end of the chute, 400 and 215 ft, respectively, remained unchanged. The rapid convergence along the ogee crest in the original design was eliminated by maintaining the desired convergence at the toe of the sidewalls along the ogee crest and projecting the batter up from this point. Tests conducted to evaluate and compare chute sidewalls battered 12V on 1H or 4V on 1H indicated no significant difference in the hydraulic performance.

19. The recommended crest and chute (type 3) and left abutment (type 9) designs increased the capacity of the structure (Plate 6) and reduced the magnitude of the standing waves traveling down the chute (Photo 5) and the depth of flow along the sidewalls. The basic spillway rating data are presented in Table 1. Water-surface

profiles along the chute sidewalls are shown in Plate 8. Vertical water depths obtained along each sidewall are presented in Table 2. Isovels and water-surface profiles along the weir axis (sta 100+16.63), sta 5+10, and at the downstream end of the chute are shown in Plate 10. At the downstream end of the chute, an average velocity of 77 fps, an average depth of 13.6 ft, and a Froude number of 3.7 were obtained for the spillway design discharge of 224,000 cfs.

Left chute sidewall

20. Due to the poor quality of the rock foundation along the downstream end of the left chute sidewall, it was structurally desirable to eliminate approximately 90 ft of the chute sidewall. Thirty-foot sections of the sidewall were incrementally removed, and observations of performance were made for the range of anticipated discharges. Current velocities and wave heights at the downstream end of the chute, along the dam embankment, in the vicinity of the outlet structure, and along the right side of the exit channel were not significantly affected by removal of 30-, 60-, or 90-ft-long sections of the chute sidewall. Flow performance with the original chute sidewall length and 90 ft of it removed (type 4) are illustrated in Photos 6 and 7, respectively.

Stilling basin

21. Tests were conducted to evaluate the effectiveness of a stilling basin located at the downstream end of the chute. A basin was installed (Figure 10) that was designed for a discharge of 133,000 cfs (standard project flood) with a minimum tailwater elevation (1032.0) equal to 85 percent of the sequent depth. Flows from 10,000 to 133,000 cfs produced undesirable eddies in the basin. These eddies were caused by the uneven distribution of flow entering the basin and the lateral flow from the left that was entrained by the jet as flow emerged from the chute. Discharges above 133,000 cfs swept the hydraulic jump from the basin, and severe waves and currents occurred downstream from the basin in the exit channel and along the highway embankment. However, the waves and currents were not as severe as those observed downstream of the recommended chute design which did not include a stilling basin. Stilling basin performance for various flows is shown in Photo 8.

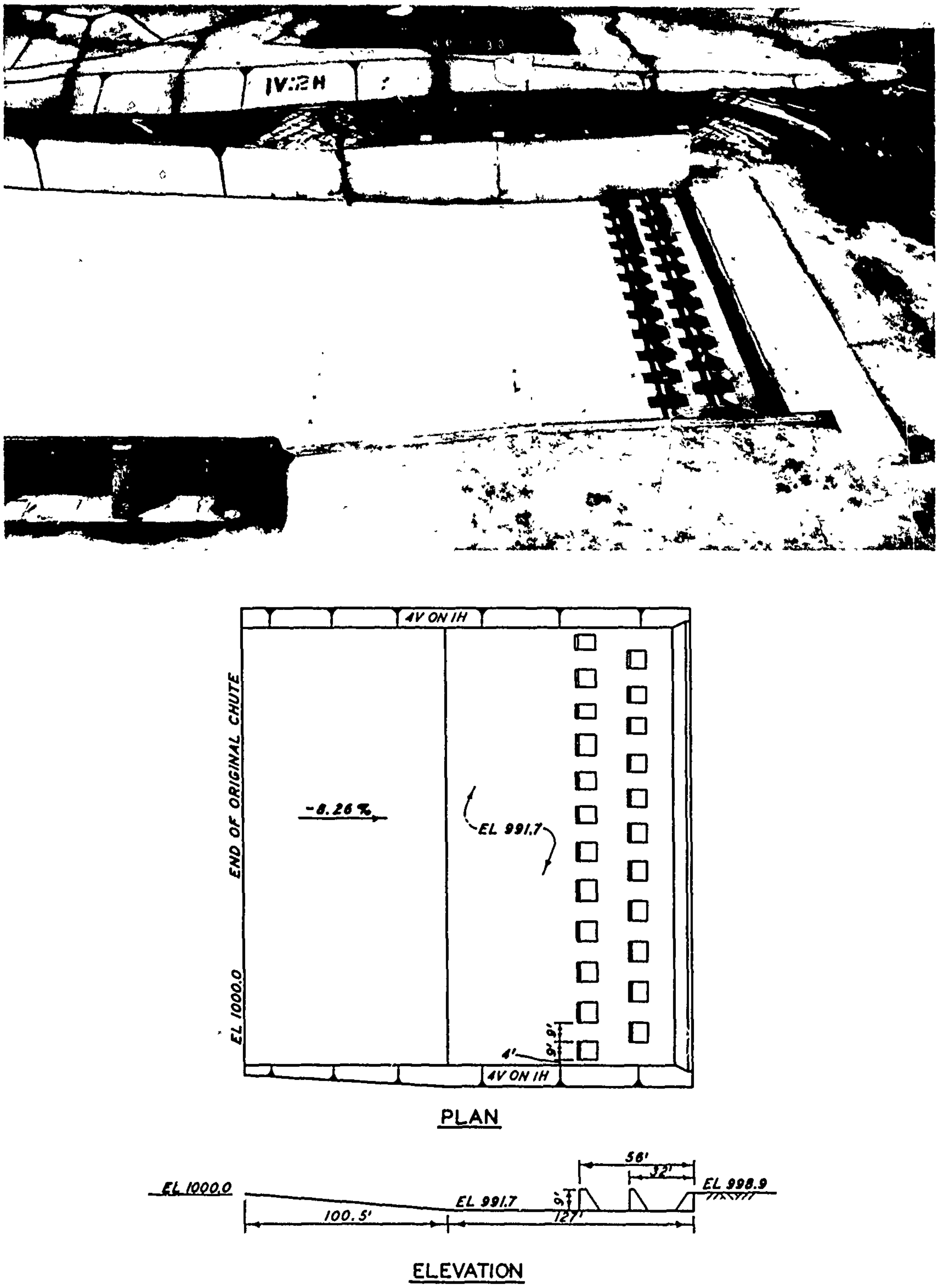


Figure 10. Stilling basin

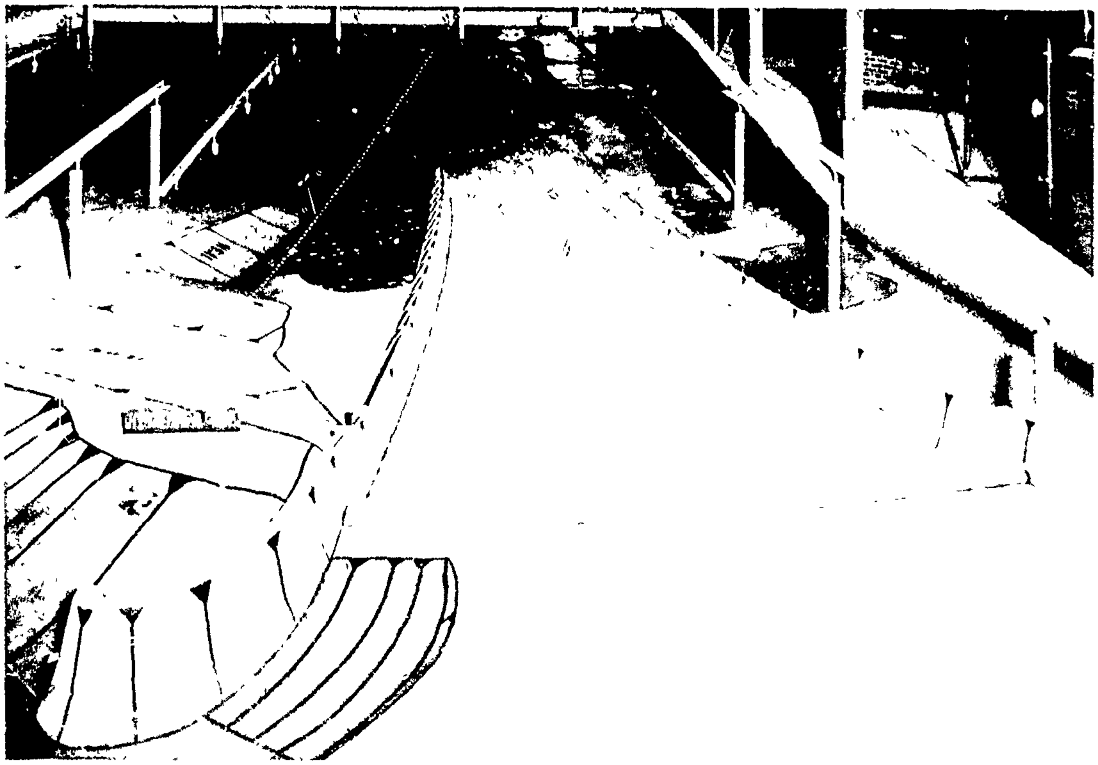
22. At a meeting held in Baltimore, Maryland, with representatives from the Office, Chief of Engineers; the U. S. Army Engineer Division, North Atlantic; and the U. S. Army Engineer District, Baltimore; it was decided that a stilling basin would not be structurally or economically feasible for this project. Furthermore, with no energy dissipator at the end of the chute, the spillway should pass the standard project flood (50,000 cfs) without extensive damage as well as the maximum probable flood (224,000 cfs) without losing the dam. Therefore, additional model tests to develop a satisfactory stilling basin were discontinued. However, the U. S. Army Engineer Waterways Experiment Station recommends the use of some type of energy dissipator at the end of the spillway chute from a hydraulic performance standpoint.

Exit Channel

23. Flow characteristics in the exit channel and along the dam embankment were similar for the original and recommended chute designs. Also, rotating the spillway had no apparent effect on the hydraulic performance of the exit area.

24. The anticipated tailwater depth at the end of the chute was not sufficient to form a true hydraulic jump for the design discharge, and a quasi-oblique jump was formed (Figure 11). Lateral flow from the left side of the exit area was entrained by the jet as it emerged from the chute and contributed to severe currents and waves along the right side of the exit channel. Velocities and wave heights along the left side of the exit area (dam embankment) were satisfactory for the range of anticipated discharges. Velocities, current directions, and wave heights for various discharges are shown in Plates 3-5.

25. The outlet works was schematically reproduced in the model. Flow conditions for a discharge of about 8,000 cfs through the outlet works exit channel are shown in Figure 12. Flow conditions for a discharge of 50,000 cfs through the chute and a combined discharge of 50,000 cfs through the chute and 9,500 cfs through the outlet works are shown in Photo 9. Operation of the outlet works for the range of anticipated discharges caused no adverse flow conditions in the exit channel.



a. Discharge 50,000 cfs, tailwater el 1021.5

b. Discharge 133,000 cfs, tailwater el 1032.0

Figure 11. Flow conditions, recommended chute design

c. Discharge 224,000 cfs, tailwater 1038.0

Figure 11 (Continued)

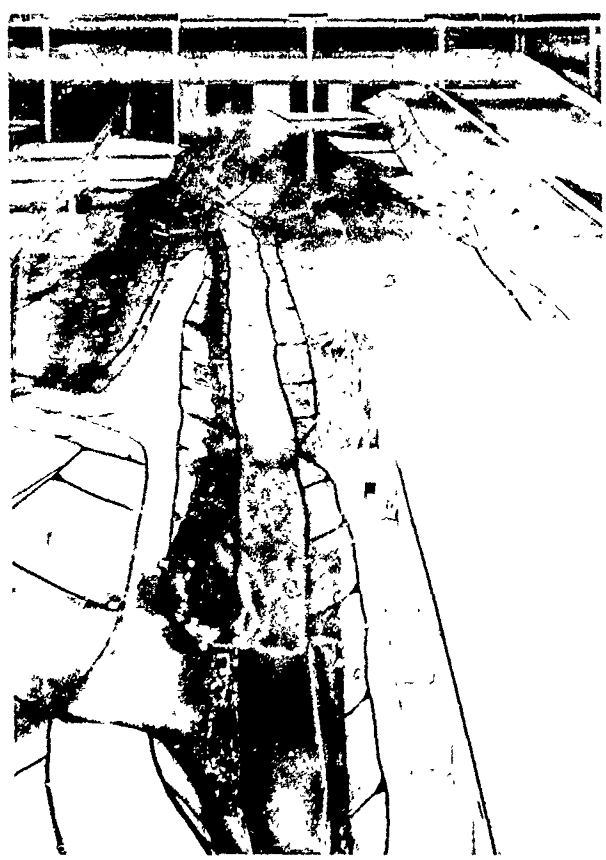


Figure 12. Flow in outlet channel; discharge 8000 cfs, pool el 1095.0, tailwater el 1006.0

PART IV: DISCUSSION

26. Approach flows to the chute spillway were tranquil for the range of anticipated discharges. Severe turbulence and water-surface drawdown observed at the left abutment was mitigated by revising the left abutment to include a combination elliptical and circular approach wall and by excavating along the inside of the wall. The streamlined approach wall provided a change in approach flow direction without excessive turbulence and drawdown. Excavating along the inside of the wall reduced the Froude number, velocities, and turbulence near the wall. Riprap with an average stone diameter of 18 in. was required for protection of the dam embankment near the upstream end of the wall. Flow performance at the right abutment was satisfactory for all flow conditions.

27. Tests indicated that the intake tower could be moved from its original position to a point as far as 150 ft downstream without adversely affecting flow performance of the spillway. Moving the tower further than 150 ft downstream created turbulence at the tower that interfered with flow over the spillway crest.

28. The convergence rate of the chute sidewalls was reduced from a ratio of 5:1 to 13:1 to attenuate the standing waves and improve the distribution of flow in the chute. To accomplish this and provide a satisfactory rock foundation, the spillway had to be rotated 0°-55'-37" in a southeasterly direction about a point near the downstream end of the chute. The vertical slope of the chute sidewalls was investigated with slopes of 4V on 1H and 12V on 1H, and no significant difference in flow characteristics was observed. The revised left abutment and chute increased the capacity of the structure, reduced the turbulence and standing waves in the chute, permitted a more even distribution of flow in the chute, and decreased the depth of flow in the chute along the sidewalls.

29. Due to the poor quality of the rock foundation at the downstream left side of the chute, 90 ft of the downstream portion of the left chute wall was removed, and the hydraulic performance of the structure was not significantly affected.

30. A stilling basin designed for a discharge of 133,000 cfs and a minimum tailwater elevation (1032.0) equal to 85 percent of the sequent depth was installed at the downstream end of the chute. Lateral flow from the left side of the exit area and uneven flow distribution in the chute contributed to the formation of eddies in the stilling basin. Discharges above 133,000 cfs swept the jump from the basin and produced severe currents and waves along the right side of the exit channel. A detailed investigation to improve the design of the stilling basin was not conducted because of a decision by the sponsor to discontinue testing of a stilling basin.

31. The tailwater depth in the exit channel was not sufficient to form a true hydraulic jump as flow exited from the chute. For discharges between 10,000 and 224,000 cfs, lateral flow from the left side of the exit channel was entrained by the jet, and severe currents and waves were created along the right side of the exit channel. Velocities and waves along the dam embankment and in the vicinity of the outlet works exit channel were not excessive.

Table 1

Basic Spillway Data Obtained from Model

Discharge cfs	Pool Elevation ft Above msl	Head on Crest ft
	Original Design	
11,000	1121.4	4.4
15,500	1122.4	5.4
19,500	1123.3	6.3
22,000	1123.8	6.8
25,000	1124.3	7.3
32,000	1125.9	8.9
45,000	1127.7	10.7
55,000	1129.2	12.2
63,000	1130.3	13.3
77,000	1132.5	15.5
89,000	1134.0	17.0
100,000	1135.3	18.3
110.000	1136.9	19.9
117,000	1137.0	20.0
123,000	1138.1	21.1
130,000	1139.4	22.4
145,000	1140.1	23.1
157,000	1141.7	24.7
165,000	1142.8	25.8
180,000	1143.8	26.8
184,000	1144.4	27.4
192,000	1145.0	28.0
	Recommended Design	
40,000	1126.1	9.1
62,000	1129.1	12.1
76,000	1131.0	14.0
93,000	1132.9	15.9
100,000	1133.7	16.7
120,000	1136.3	19.3
134,000	1137.2	20.2
156,000	1139.6	22.6
180,000	1141.5	24.5
200,000	1144.0	27.0
224,000	1146.0	29.0

Table 2

Water Depths, Type 3 Crest and Chute
Discharge 224,000 cfs

Horizontal Distance Along and from Station 100+32.05 ft	Vertical Water Depth, ft Left Side	Right Side
0	17.0	20.5
10	14.3	19.7
20	15.3	19.4
40	14.8	19.0
50	12.3	17.7
70	10.7	15.4
85	10.5	15.4
135	10.2	15.5
235	10.3	15.4
335	9.2	15.5
435	9.2	16.9
535	9.6	17.0
635	10.8	17.2
735	10.9	16.1
835	12.5	14.3
935	15.2	13.1
1035	16.1	11.5
1135	16.8	11.7

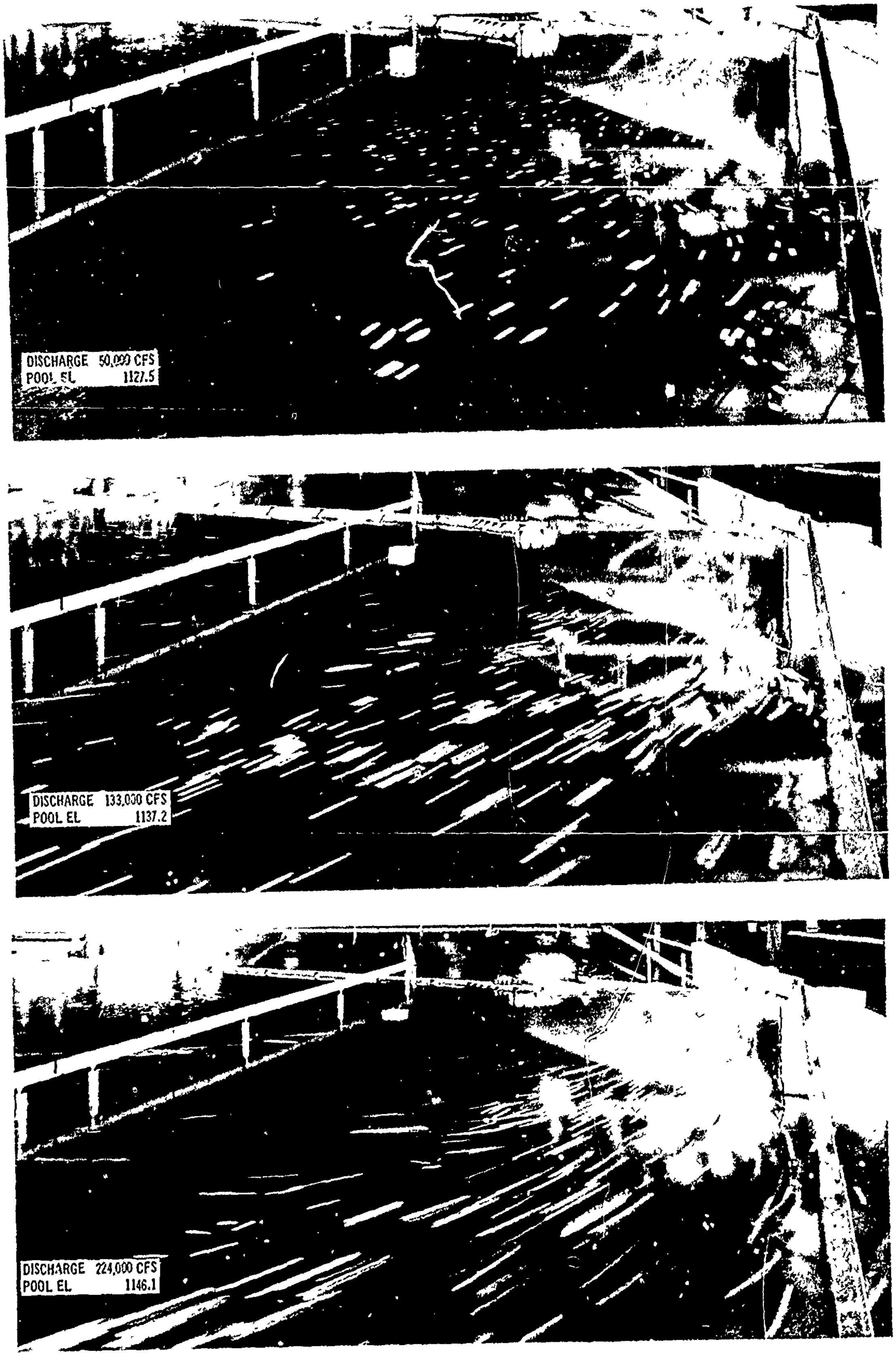
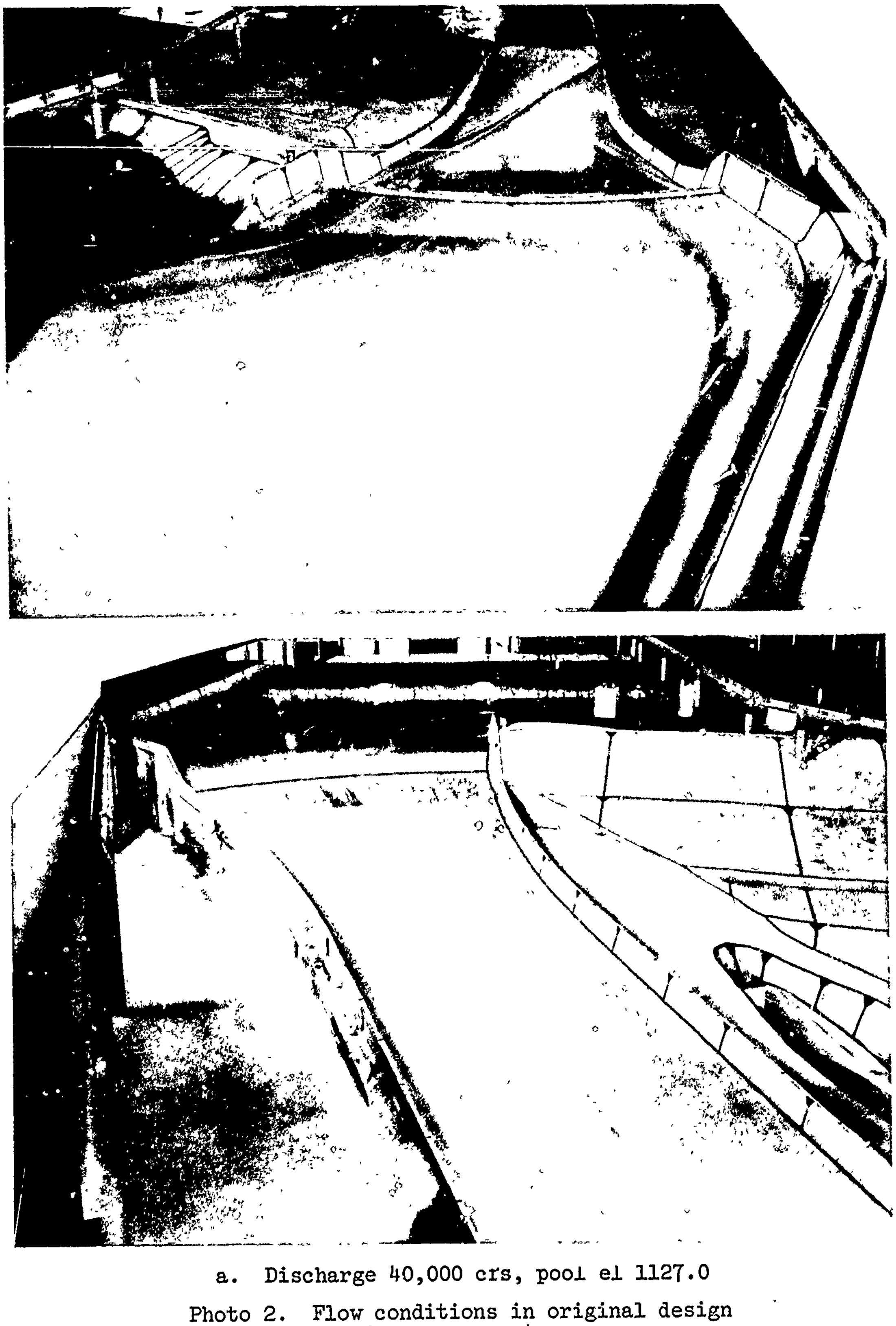
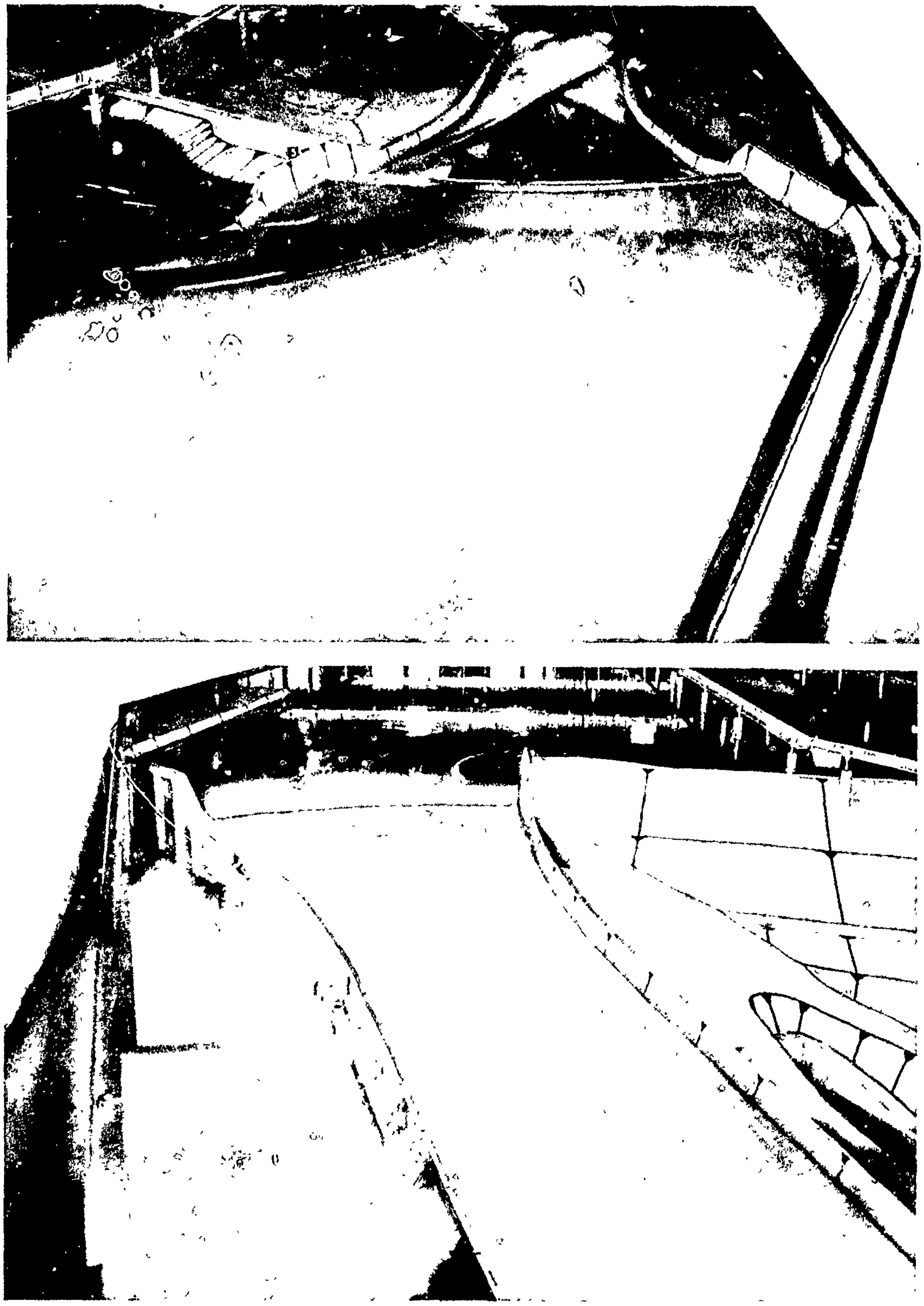


Photo 1. Approach flows



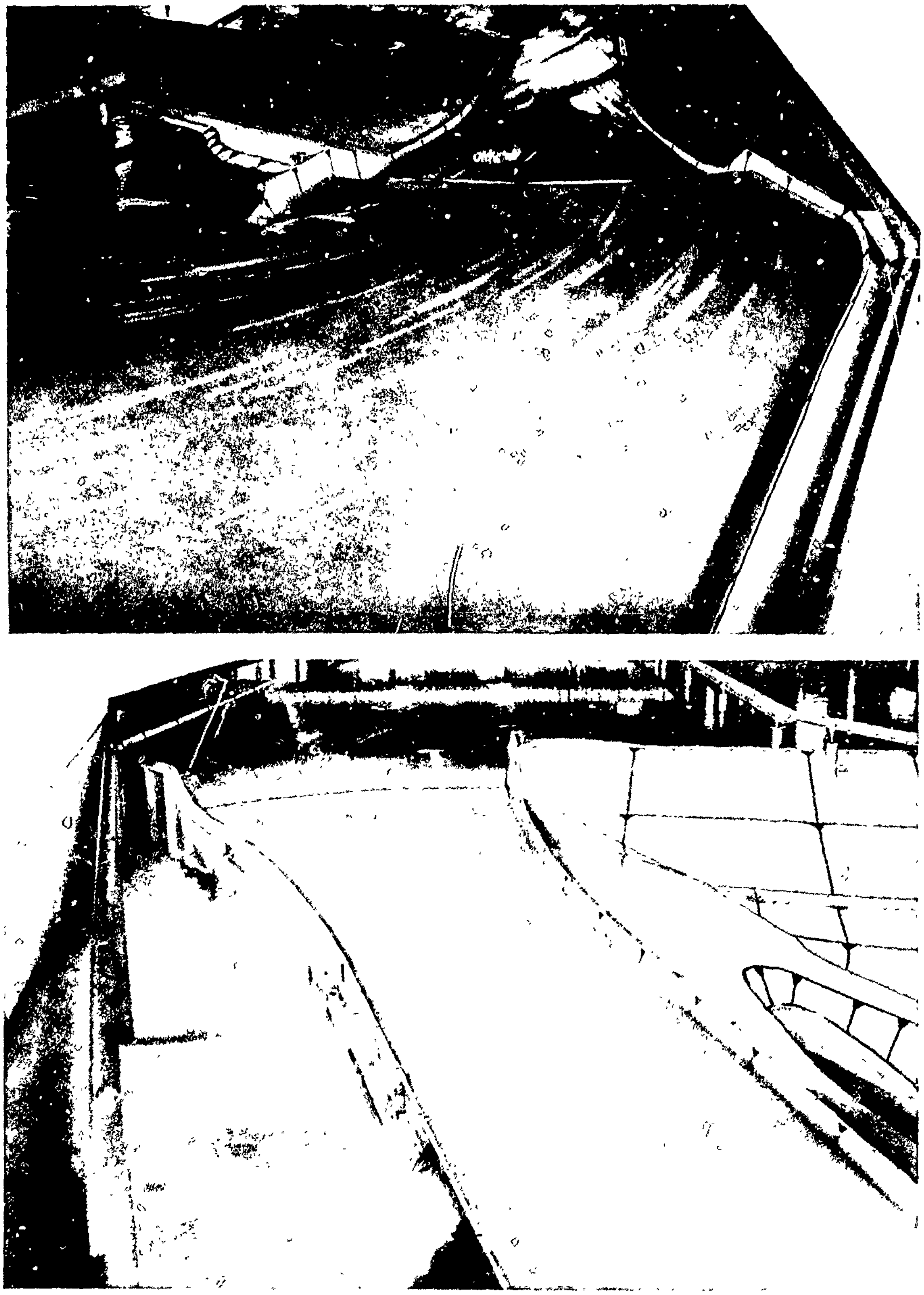
a. Discharge 40,000 cfs, pool el 1127.0

Photo 2. Flow conditions in original design
(sheet 1 of 3)



b. Discharge 120,000 cfs, pool el 1137.7

Photo 2 (sheet 2 of 3)



c. Discharge 200,000 cfs, pool el 1146.0

Photo 2 (sheet 3 of 3)

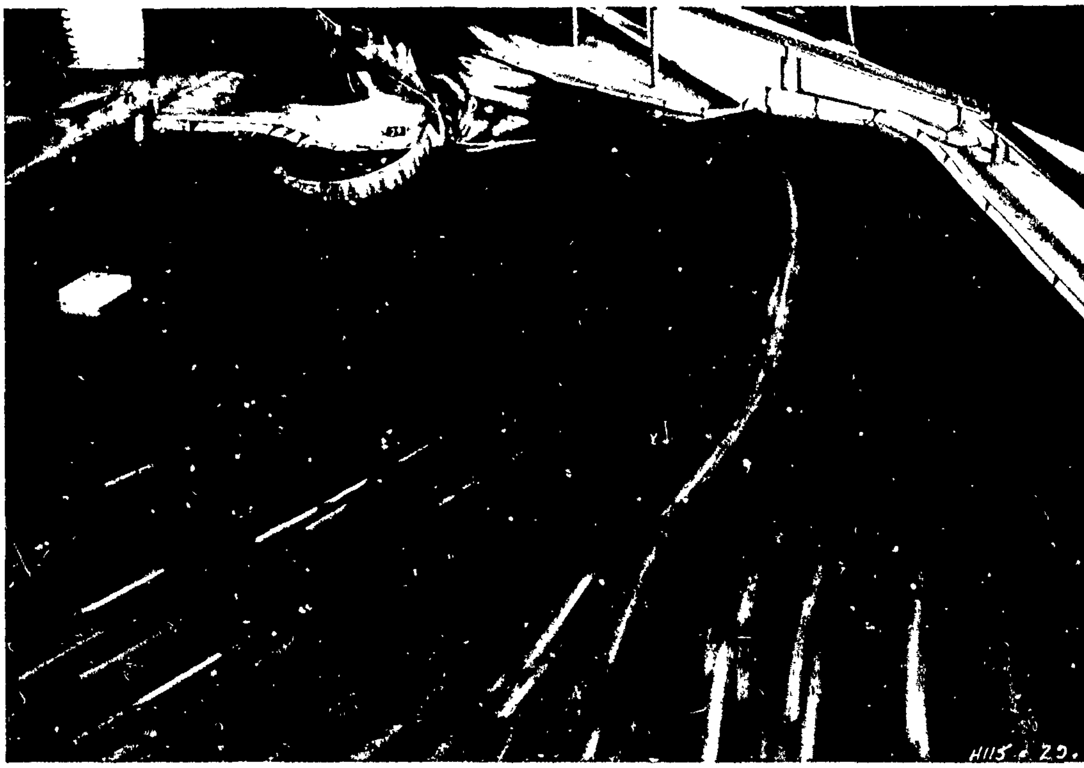
a. Discharge - 40,000 cfs, tailwater el 1020.6

b. Discharge 120,000 cfs, tailwater el 1031.0

Photo 3. Flow conditions at downstream end of chute, original design (sheet 1 of 2)

c. Discharge 200,000 cfs, tailwater el 1035.9

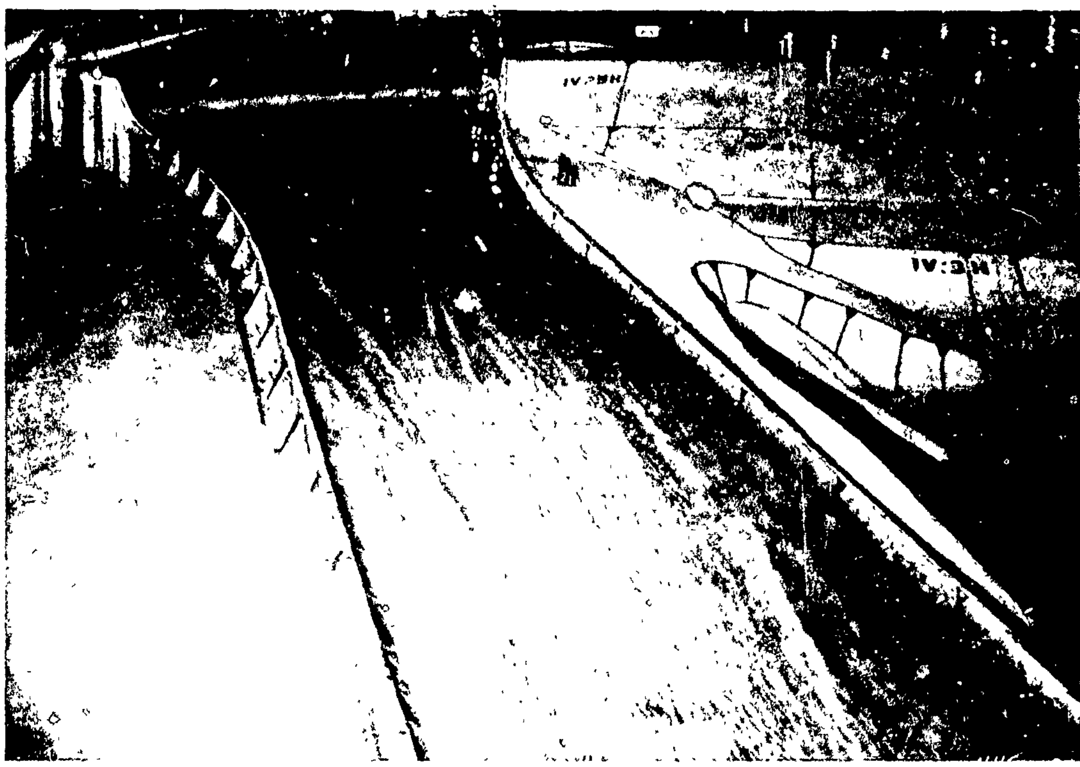
Photo 3 (sheet 2 of 2)



a. Approach

b. Left abutment

Photo 4. Flow conditions, type 4 left abutment; discharge 200,000 cfs, pool el 1144.7 (sheet 1 of 2)



c. Chute

Photo 4 (sheet 2 of 2)

a. Discharge 40,000 cfs

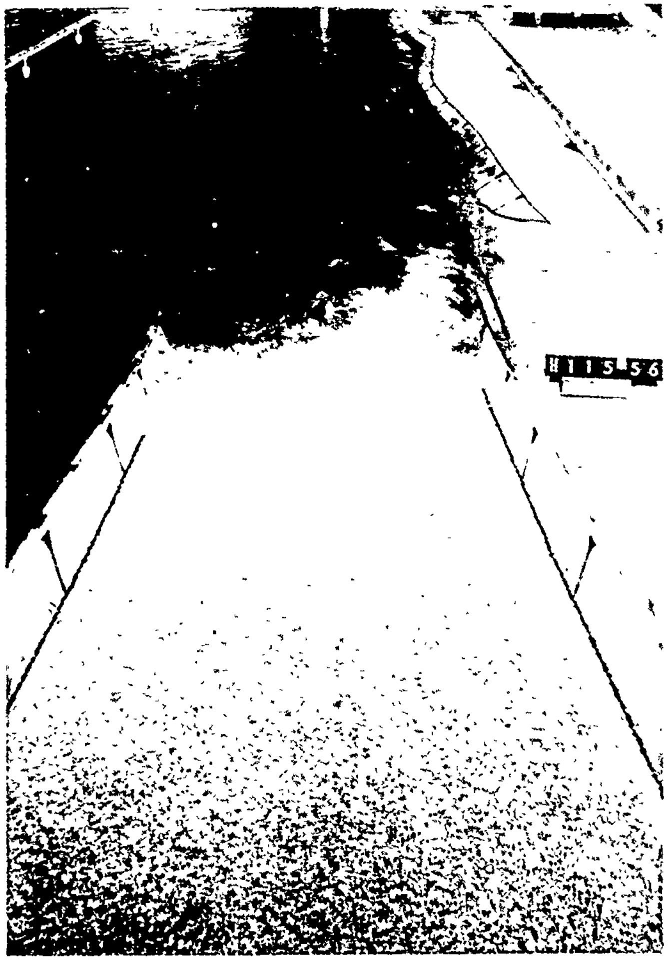
b. Discharge 80,000 cfs

Photo 5. Flow conditions for recommended crest, chute, and abutment (sheet 1 of 2)

c. Discharge 160,000 cfs

d. Discharge 224,000 cfs

Photo 5 (sheet 2 of 2)



a. Discharge 50,000 cfs, tailwater el 1021.5

b. Discharge 224,000 cfs, tailwater el 1038.0

Photo 6. Original left chute sidewall flow conditions

a. Discharge 50,000 cfs, tailwater el 1021.5

b. Discharge 224,000 cfs, tailwater el 1038.0

Photo 7. Type 4 left chute sidewall flow conditions

a. Discharge 133,000 cfs, tailwater el 1032.0

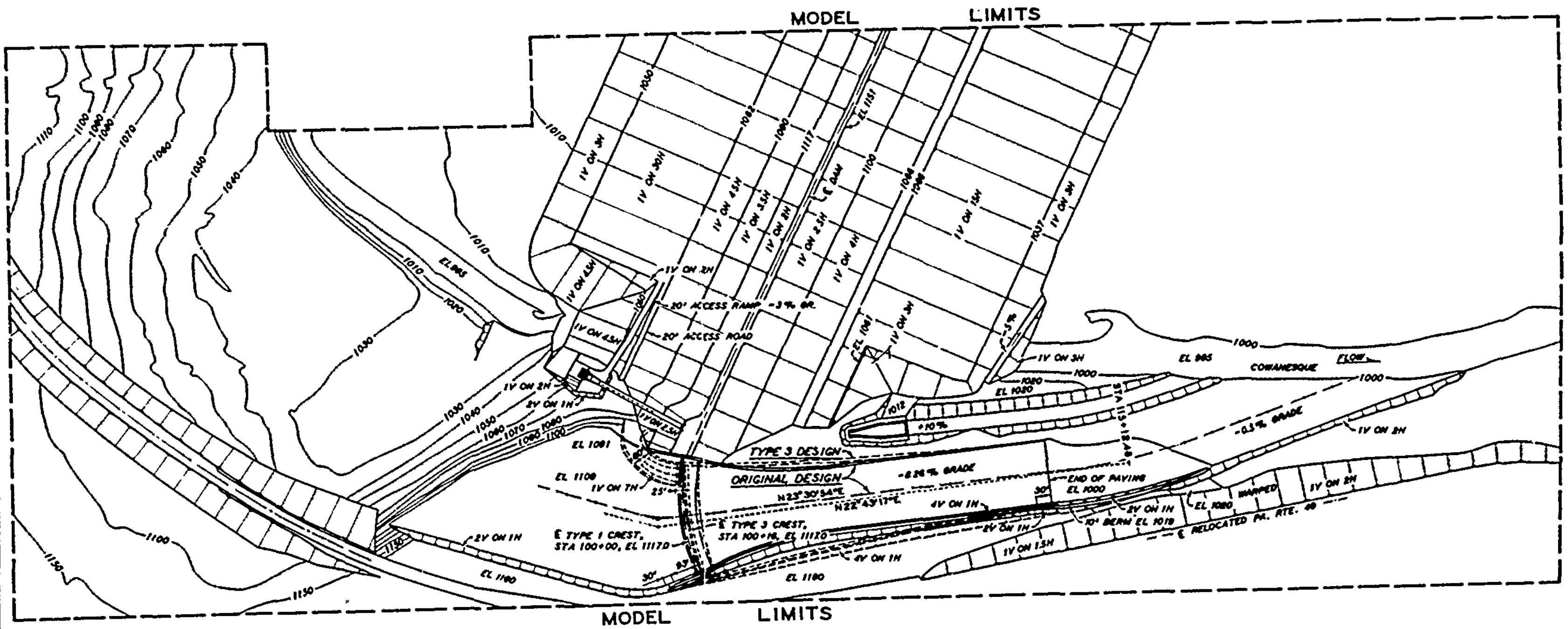
b. Discharge 224,000 cfs, tailwater el 1038.0

Photo 8. Stilling basin flow conditions

a. Discharge 50,000 cfs (chute), tailwater el 1021.1

b. Discharges 9,500 (outlet structure) and 50,000 cfs (chute), tailwater el 1024.0

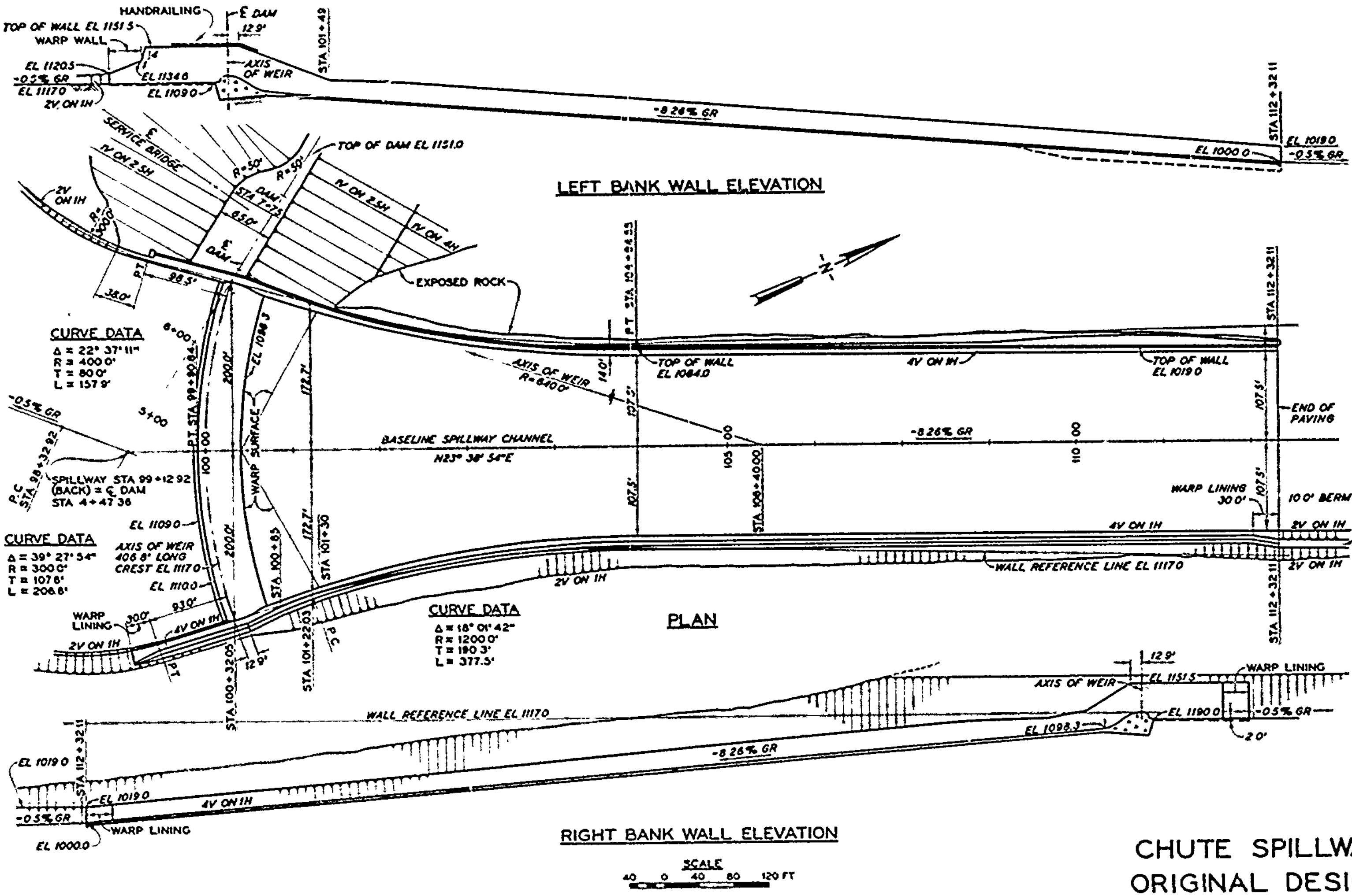
Photo 9. Exit and outlet channel flow conditions



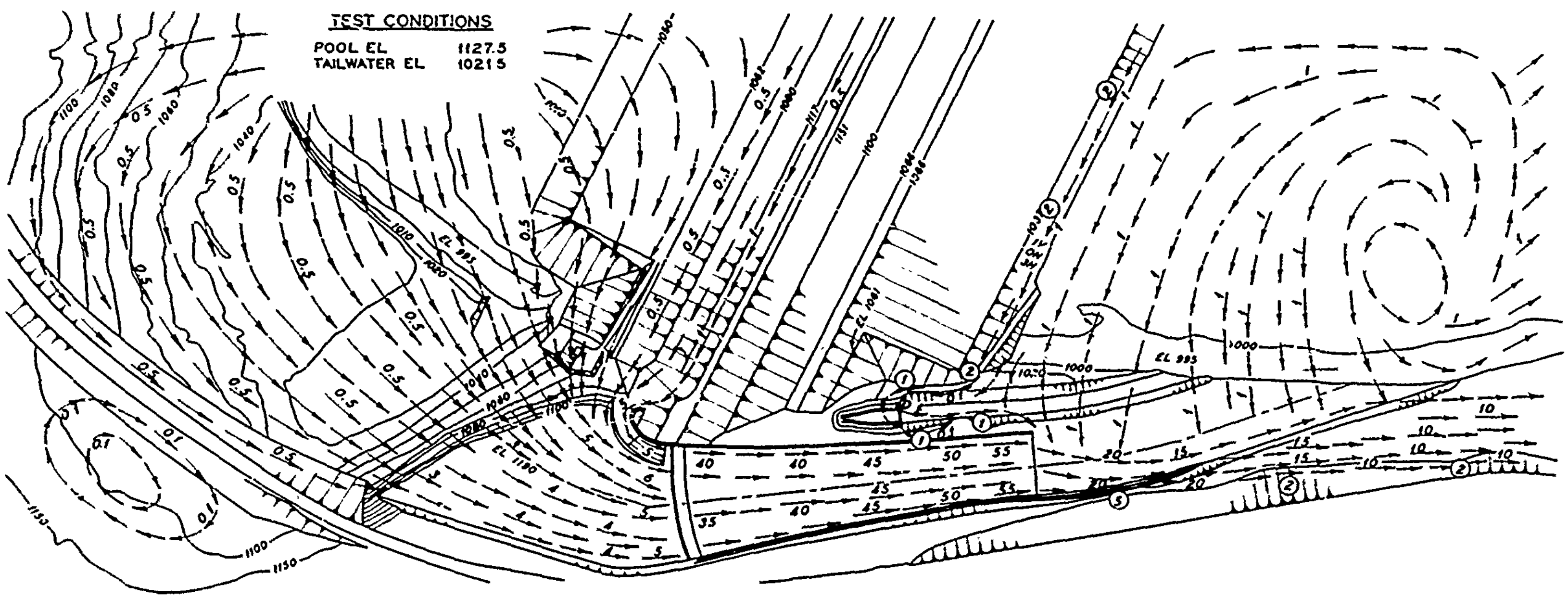
GENERAL PLAN OF PROJECT
ORIGINAL AND TYPE 3 DESIGNS

SCALES IN FEET

PLATE 1



CHUTE SPILLWAY
ORIGINAL DESIGN



LEGEND

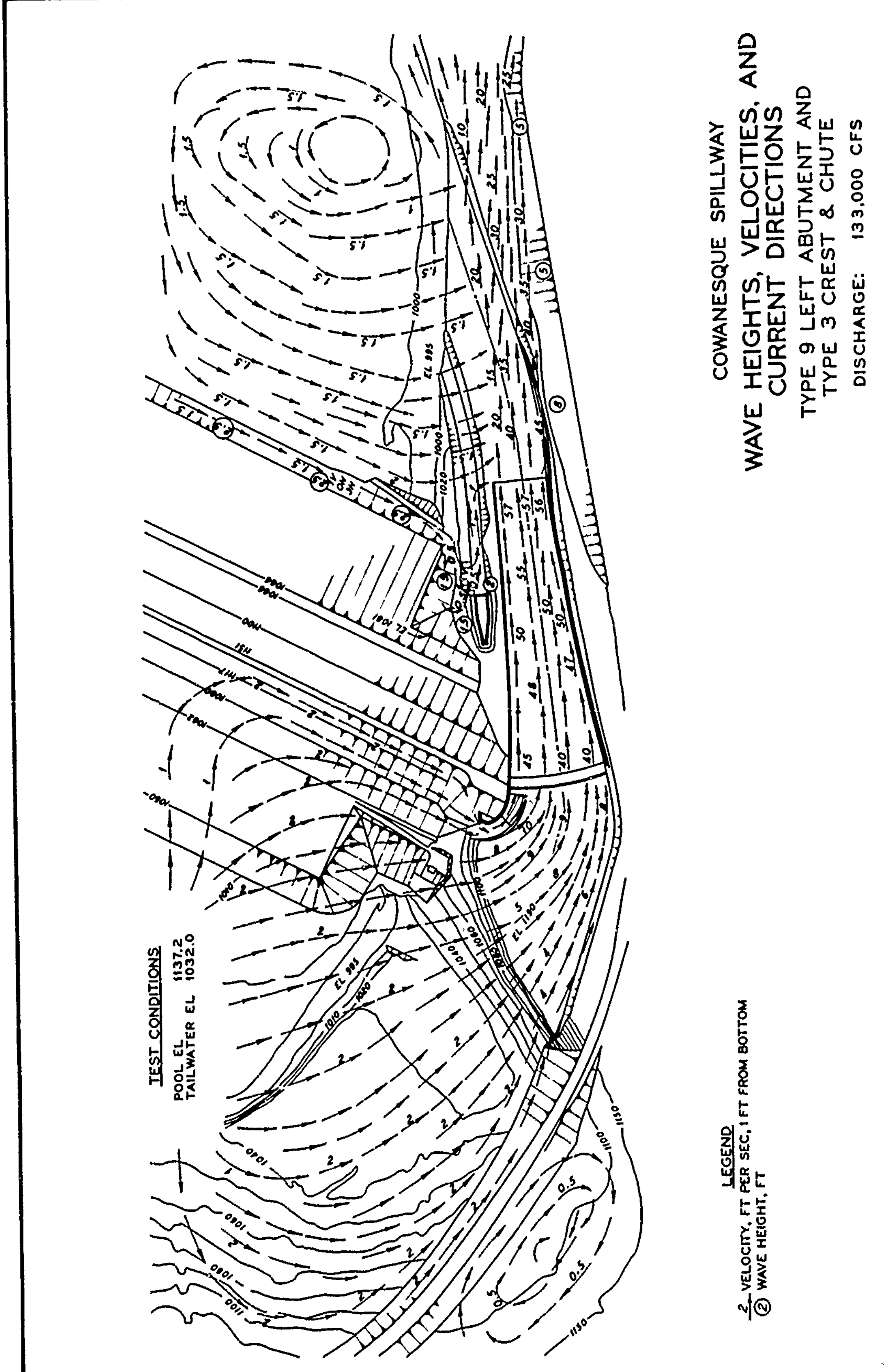
0.5 VELOCITY, FT PER SEC, 1 FT FROM BOTTOM
(2) WAVE HEIGHT, FT

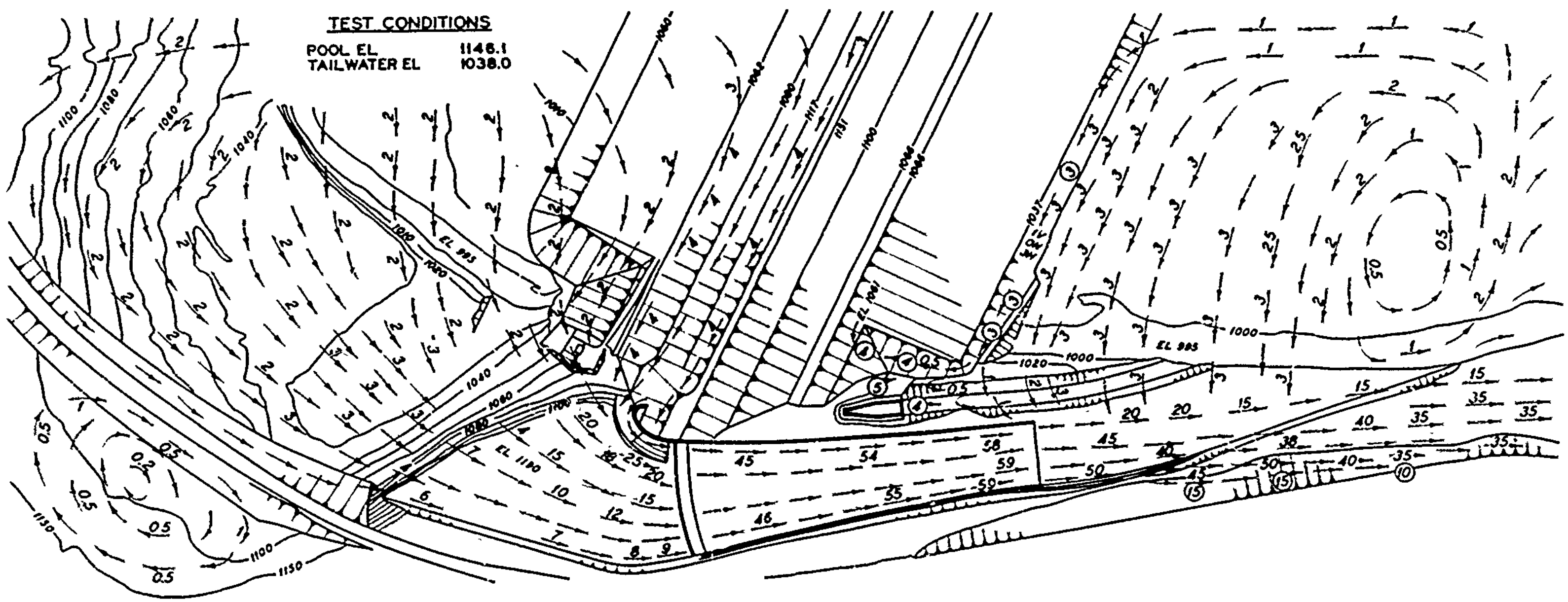
COWANESQUE SPILLWAY

WAVE HEIGHTS, VELOCITIES, AND CURRENT DIRECTIONS

TYPE 9 LEFT ABUTMENT AND TYPE 3 CREST & CHUTE

DISCHARGE: 50,000 CFS


COWANESQUE SPILLWAY
WAVE HEIGHTS, VELOCITIES, AND CURRENT DIRECTIONS
TYPE 9 LEFT ABUTMENT AND TYPE 3 CREST & CHUTE
DISCHARGE: 133,000 CFS
TEST CONDITIONS
POOL EL 1137.2
TAILWATER EL 1032.0
LEGEND
2 VELOCITY, FT PER SEC, 1 FT FROM BOTTOM
② WAVE HEIGHT, FT



LEGEND
2 VELOCITY, FT PER SEC, 1 FT FROM BOTTOM
(3) WAVE HEIGHT, FT

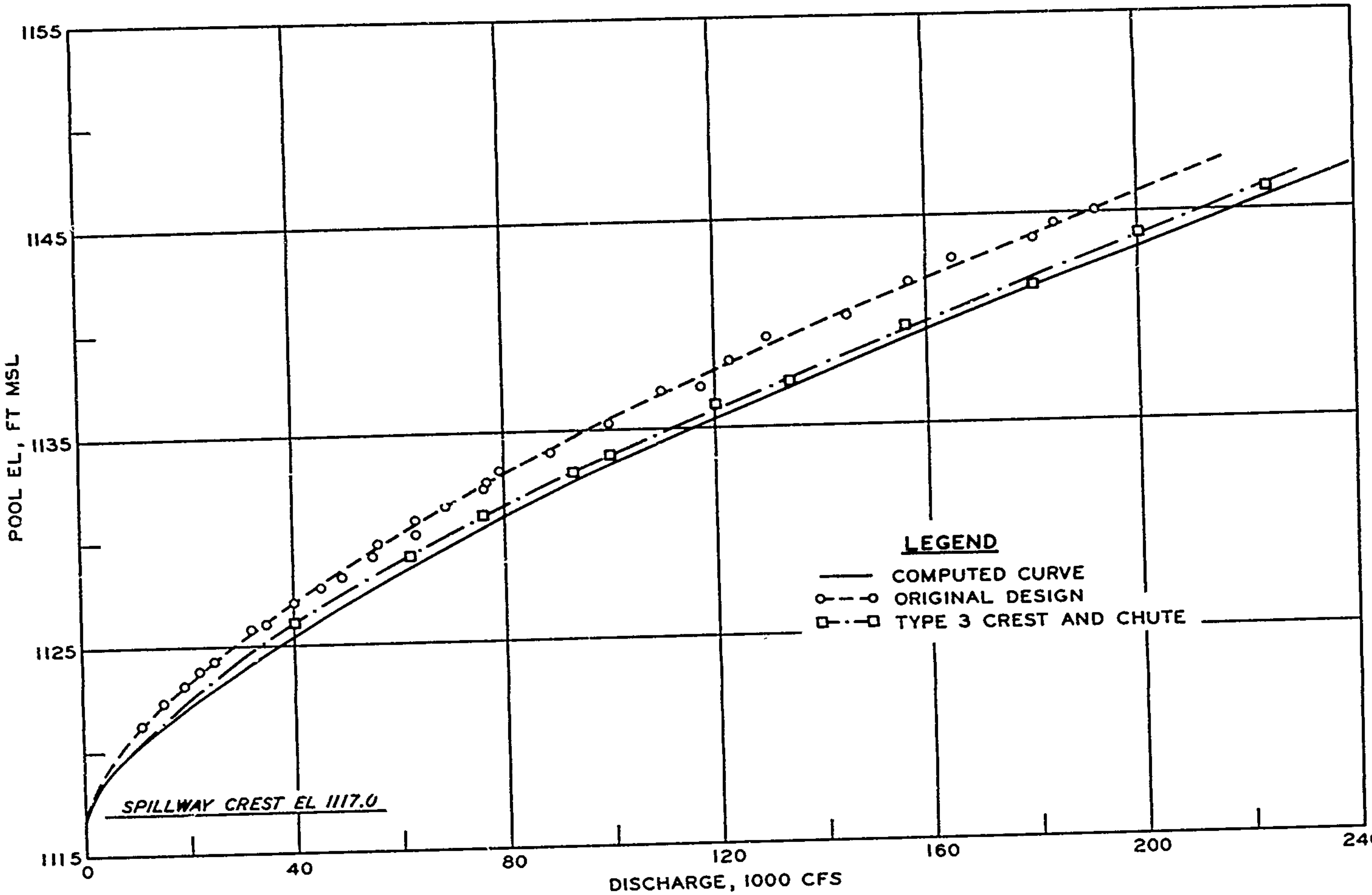
COWANESQUE SPILLWAY

WAVE HEIGHTS, VELOCITIES, AND CURRENT DIRECTIONS

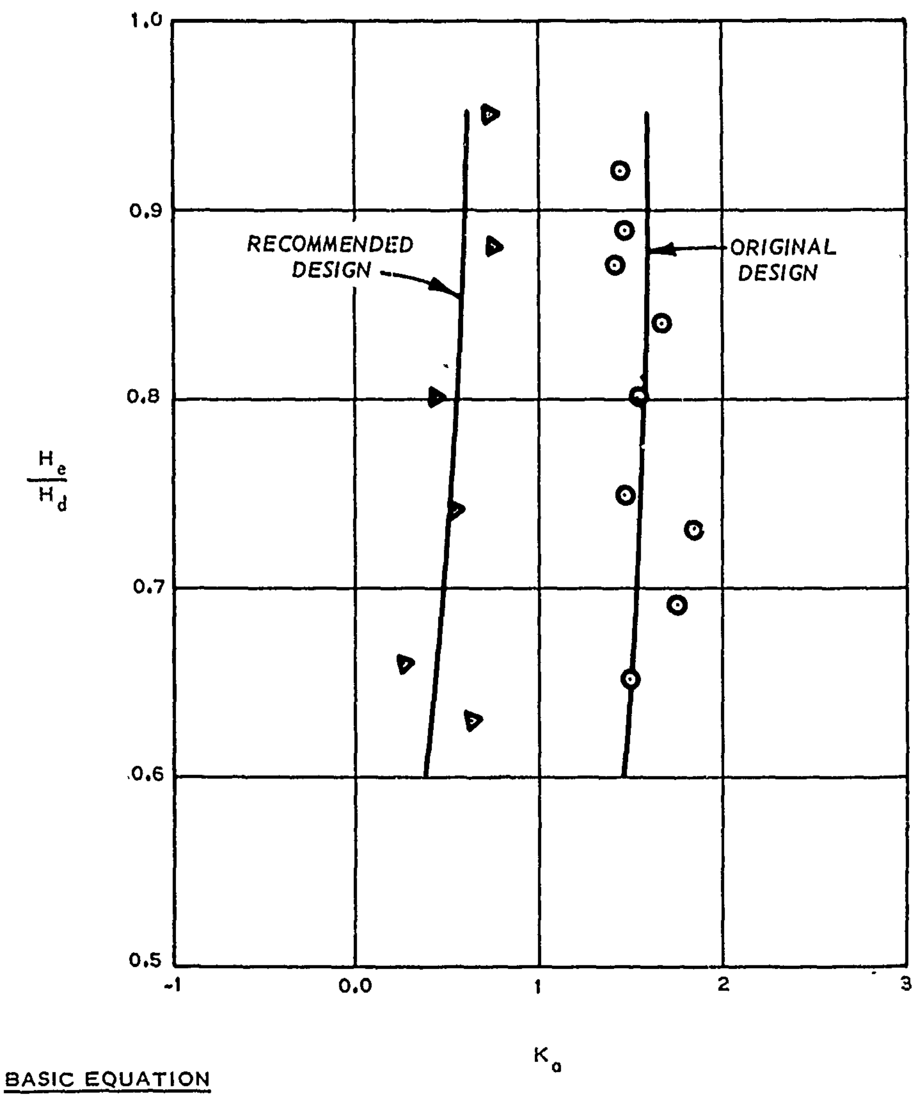
TYPE 9 LEFT ABUTMENT AND TYPE 3 CREST & CHUTE

DISCHARGE: 224,000 CFS

PLATE 5



SPILLWAY RATING CURVES



BASIC EQUATION

$$Q = C\left[L - 2(K_a)(H_e)\right]H_e^{3/2}$$

WHERE

Q = DISCHARGE, CFS

C = DISCHARGE COEFFICIENT

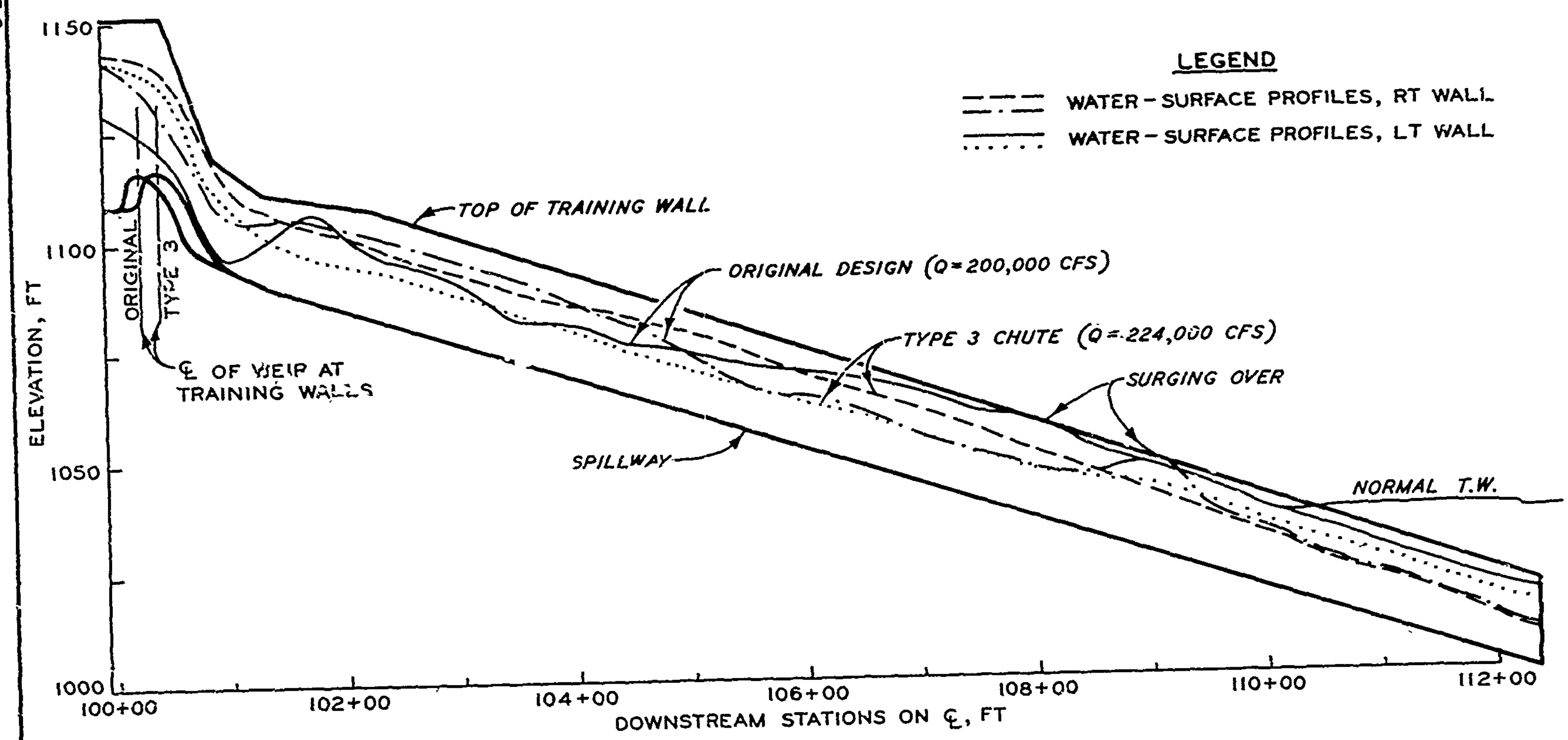
L = NET LENGTH OF CREST

K_a = ABUTMENT CONTRACTION COEFFICIENT

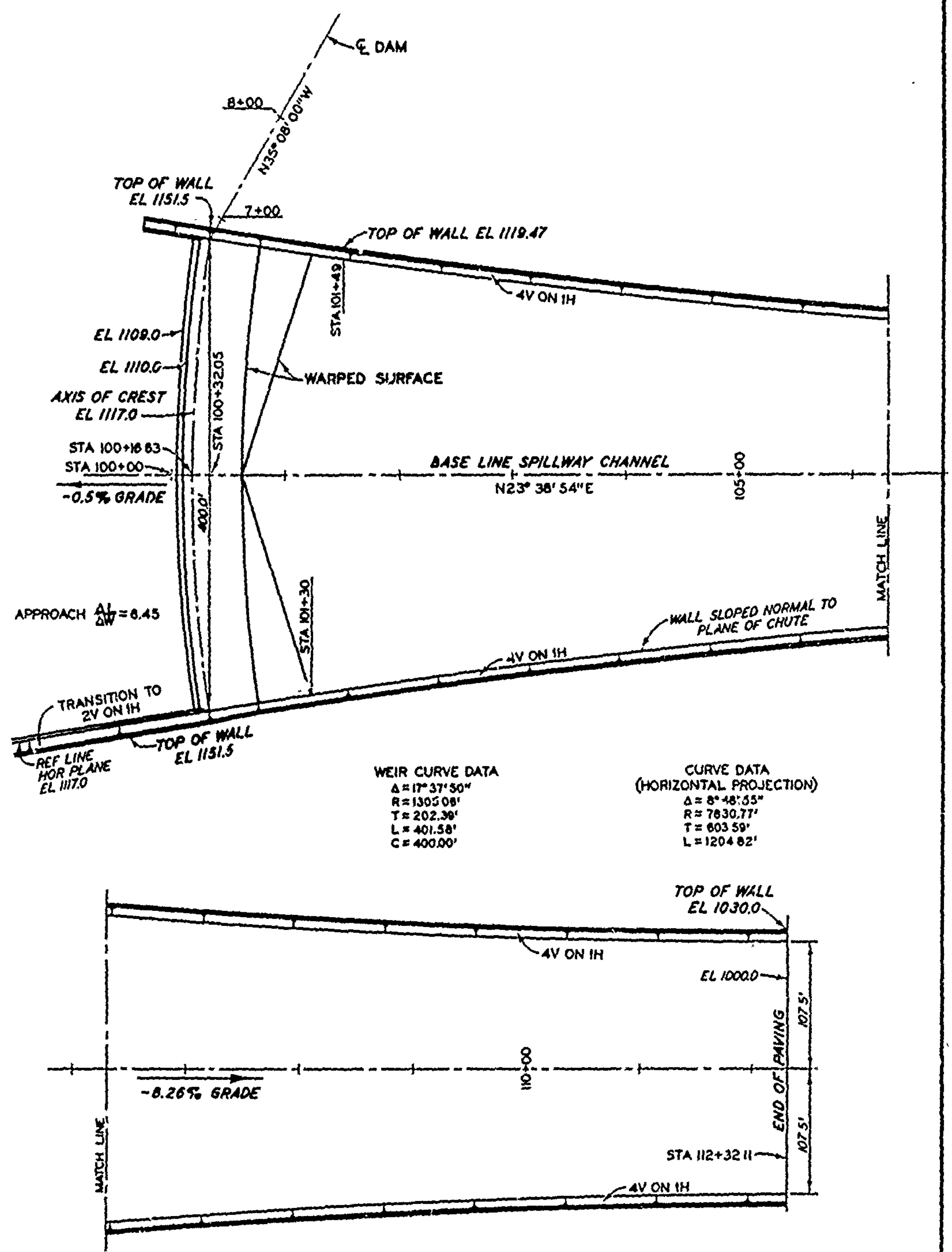
H_e = TOTAL HEAD ON CREST, FT

H_d = 30.6 FT (DESIGN HEAD)

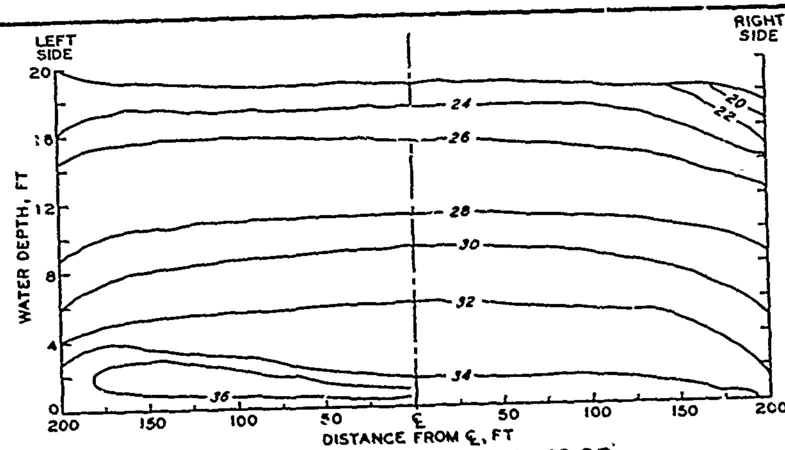
ABUTMENT CONTRACTION COEFFICIENTS



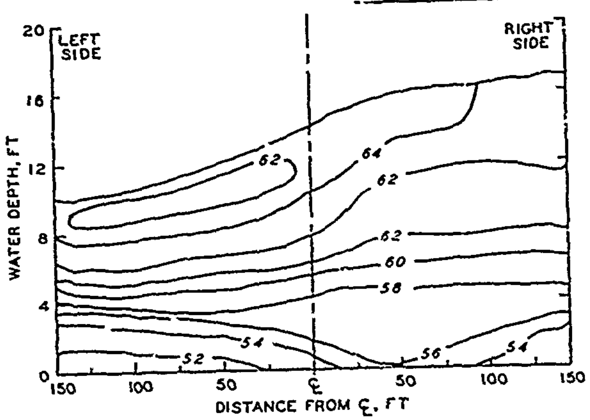
WATER-SURFACE PROFILES
ORIGINAL AND RECOMMENDED DESIGNS



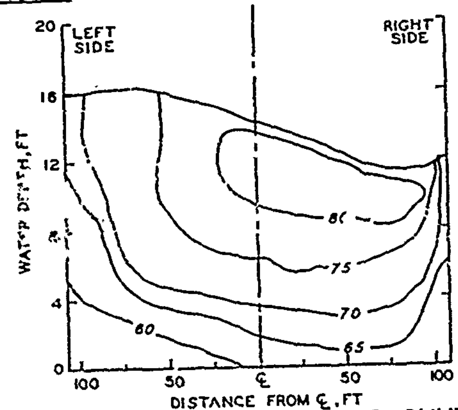
TYPE 3
CRESS & CHUTE



a. WEIR AXIS–STA 100+16.63



b. STA 5+10



c. DOWNSTREAM END OF CHUTE

NOTE VELOCITIES ARE IN PROTOTYPE FEET PER SECOND

ISOVELS
RECOMMENDED CHUTE
TYPE 9 LEFT ABUTMENT
DISCHARGE 224,000 CFS

In accordance with ER 70-2-3, paragraph 6c(1)(b), dated 15 February 1973, a facsimile catalog card in Library of Congress format is reproduced below.

Fletcher, Bobby P
Chute spillway for Cowanesque Dam, Cowanesque River, Pennsylvania, hydraulic model investigation, by Bobby P. Fletcher. Vicksburg, U. S. Army Engineer Waterways Experiment Station, 1976.
1 v. (various pagings) illus. 27 cm. (U. S. Waterways Experiment Station. Technical report H-76-12)
Prepared for U. S. Army Engineer District, Baltimore, Baltimore, Maryland.

1. Chute spillways. 2. Cowanesque Dam. 3. Hydraulic models. I. U. S. Army Egineer District, Baltimore. (Series: U. S. Waterways Experiment Station, Vicksburg, Miss. Technical report H-76-12)
TA7.W34 no.H-76-12